**Golder Associates**
CONSULTING GEOTECHNICAL AND MINING ENGINEERS

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# REGIONAL STUDY OF COAL MINE WASTE DUMPS IN BRITISH COLUMBIA

## STAGE II

## VOLUME I
## TEXT

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# 1.0 INTRODUCTION

## 1.1 Background

This report presents the results of a study of the behaviour of waste rock dumps associated with open pit coal mining operations in the Rocky Mountain region of British Columbia and Alberta. The coal measures occur within Cretaceous Sedimentary beds, and the waste rock dumps comprise siltstones, sandstones, mudstones and shales from the overburden and interseam beds.

There are currently seven major coal mining operations in British Columbia, from which the combined production of clean coal is estimated to be in the range of 30 to 35 million tonnes per year. The associated production of waste rock is estimated to be about 150 million tonnes, or approximately 100 million loose cubic metres of material per year.

Current practice is to dump the waste material as close as practicable to the open pit where it is generated. Major differences in elevation between the pit and the dump would add significantly to the hauling cost. In the interest of economics, the waste material is consigned to dumps located as close as practicable to the open pits many of which are located at high elevations on the mountain slopes. In many instances waste rock dumps are developed over relatively steep terrain.

Figure 1.1 shows the rate of occurrence of significant failure events during the period 1970 to 1986. There appears to be a relationship between the increased number of operations which have come on stream during the 1980's and the occurrence of major dump failure events.

Golder Associates have participated in the design and stability assessments and in the monitoring of the performance of waste dumps at most of the coal properties during the 1970's and 1980's. This experience has led to a gradual evolution in our understanding of the mechanisms which control the behaviour of major waste dumps under a variety of conditions. Two key questions have arisen, particularly since 1980, with the increase in production and number of mines in operation, but reinforced by decreasing prices, increased competition and reducing demand.

- Issue No. 1. Is the increase in incidence of failures merely the reflection of increased mining activity, or are a new set of circumstances emerging which are not well understood or adequately dealt with by current technical and operating practice?
- Issue No. 2. Can all waste dumps be considered in the same way by designers, regulators and operators?

## 1.2 Development of Study Program

With these questions in mind, Golder Associates formulated the framework for an unsolicited proposal to study existing coal mine waste dumps in British Columbia. Preliminary discussions were held with the operating mines to explain the concept of the proposed study. Unanimous support for the study concept was received from the industry, and as a result, a more detailed outline was submitted to the British Columbia Ministry of Energy, Mines and Petroleum Resources. During August and September, 1986, further discussions of the concerns and issues raised by both the coal mining industry and the regulatory authorities were held to more clearly define the issues.

Stage I of the study was commissioned by the Ministry of Energy Mines and Petroleum Resources (MEMPR). The purpose of Stage I was to coordinate the concerns and input of both the coal mining industry and MEMPR to formulate a detailed proposal for study of mine waste dumps.

On October 6, 1986, an ad hoc committee meeting was held. This meeting was attended by representatives from six coal mines, the Engineering and Inspection Branch of MEMPR, and by Golder Associates. Objectives of the proposed study were reviewed, and Golder Associates was requested to proceed with an application to obtain funding for the study.

On November 12, 1986, an application was submitted to the Canada-British Columbia Mineral Development Agreement. By December, 1986, an agreement in principle was reached for a funding arrangement for the study. Funding of the study has been shared by the following organizations:

- Canada-British Columbia Mineral Development Agreement (C-BCMDA)
- Federal Panel on Energy Research and Development (PERD) through the Department of Supply and Services Contract Number 23440-6-9188/01SQ. This funding was arranged in cooperation with the Surface Mining Laboratory of the Canada Centre for Mineral and Energy Technology of Energy, Mines and Resources, Canada (CANMET)
- Seven participating coal mines in British Columbia and one in Alberta
- Mining Association of British Columbia

The industry participants have contributed both funds and manpower equivalent in value to funding provided by the C-BCMDA and PERD.

In the latter part of December 1986 and the early part of January, 1987 visits were made to each of the seven participating coal mine sites to initiate collection of data.

In February, 1987, Smoky River Coal Limited of Grand Cache, Alberta, agreed to participate in the study.

We wish to thank the participating coal mines for their cooperation in making information available to us. We have appreciated the opportunity to carry out this interesting project.

## 1.3 Qualification

It is noted that the nature of waste rock piles studied in this report may be very specific to the sites involved. The future use of any graphical representations and numerical criteria presented in this report should be confirmed by site specific study.

## 2.0 EXECUTIVE SUMMARY

Available data on performance and contributing factors for eight coal mines have been reviewed. The following is a summary of key conclusions:

- o Failure frequency

  Annual rate of occurrence of failure events and periods of high rates of deformation have increased markedly in the 1980's corresponding to increased number of operations and less favourable conditions.

- o Site similarity

  General similarity exists between the mine sites studied for the following parameters;

  - foundation materials
  - foundation topography
  - weather conditions

- o Variables affecting risk of failure

  Using available information we have carried out simple correlations to test for the presence of dominant factors affecting the behaviour and incidence of failure in waste dumps. We consider that the results indicate the following factors significantly affect incidence of failure or increased rates of crest deformation:

  - Waste rock quality when a high proportion of fine grained particles are present, compounded by seepage.

  - Increased rates of dumping generally result in increased rates of crest deformation. However, inadequate documentation of waste rock quality has precluded development of clearly defined relationships.

  - Precipitation events in excess of about 20 - 30 mm in 24 hours appear to result in increased rates of crest deformation.

  - Foundation toe slopes in excess of about 25 degrees result in a significant increase in failure events. This finding is in conformance with indicated factors of safety close to unity.

- Appropriate control of direction of crest advance in relation to topographic features such as deeply incised gullies can result in significant increases in the factor of safety against base sliding.

- Rapid loading of loose saturated fine grained foundation soils results in the toe spreading. The extent of failure and runout distance is variable, and depends upon topography and soil conditions in the runout zone.

- Development of excess pore pressure within dumps due to the presence of poorly draining fine-grained material may result in large and relatively rapid failure. This cause of failure is not frequent. Build-up of hydrostatic pressure beneath frozen ground appears to be a factor that can contribute to failure.

Clearly the inter-relationship between the various factors can be complex. We have not been able to develop a mathematical model which simulates the relative importance of the various factors because the available information does not contain a sufficient number of complete data sets. In general data sets for specific dumps lacked detailed information on waste rock quality and rate of dumping. Explanations of post failures tended to be generalized and are not supported by detailed investigation of the preceding events. Thus, we recommend that in future the mines accumulate information on the above noted factors on a routine basis. Periodic review of this information is expected to provide valuable feedback for planning purposes. Also, we expect the potential relationships between behaviour and contributing factors established in this report will become more clearly defined.

In answer to issue No. 1 raised in Section 1.1., the available information does not allow us to conclude that any particular factor(s) have become more dominant in recent years. Further study based on more complete sets of data is required in order to clarify this issue. We consider that comparison of behaviour at different sites is beneficial provided that each situation is completely characterized. Common

threads of behaviour and conditions do exist between the mine sites studies but we should continue to seek explanations for differences.

- o Monitoring

  In reviewing the data provided by the eight mines we have the following comments on monitoring procedures:

  - Visual monitoring of crest and face deformations is an essential element of safe dump operation. Total reliance on monitor measurements is not advisable.

  - Wireline monitors are a proven satisfactory means of obtaining data to determine trends of accelerating crest displacements and these trends provide advance warning of impending failure.

  - Alternatively, survey measurements of crest displacement provides sufficient accuracy but is not continuously available.

  - Care is required in the proper positioning of monitors. Some operational personnel need to be convinced that although monitor setups on constricted dump platforms may be inconvenient, they are nevertheless important.

  - Overall accuracy of wireline measurements is satisfactory, and taking a running average of three consecutive readings appears to be a good approach to smoothing the indicated trends. Increased future use of computers may result in more sophisticated processing and analysis of trends.

  - Frequency of measurement should be a function of dump performance, e.g. readings 1 or 2 times per shift for a relatively stable large dump increasing to hourly for dumps experiencing high rates of displacement. The graduated approach to frequency of monitoring adopted by several operations appears to work well. Periodic review of monitor data may result in a modified frequency.

  - Almost without exception plots of rate of crest displacement versus time provide ample indication of accelerating trends which may precede a failure. Plots of cumulative displacement with time also may be practical.

- Setting caution limits for dumping short and pushing, or for ceasing dumping should be conservative during the early stages of development of any major new dump. Careful documentation of behaviour in the initial stages of development should permit rational review and potential modification of these caution limits.

The emerging philosophy of mine operations for periodic review of dump performance and monitoring procedures should be encouraged. Reliable documentation of contributing factors will facilitate this process.

- The routine documentation of material quality and rate of consignment should be encouraged as an integral part of dump monitoring. Also observations of dump face and platform deformation corresponding to this documentation are very important in identifying different mechanisms. To date, very few well documented cases are available.

  A standardized guide and checklist for documentation of contributing factors should be developed. The guidelines should be simple and practical.

o Analysis

- Available evidence in the form of field measurements, model tests and analytical sensitivity analyses indicate that failure mechanisms can be treated as a number of wedges. Typically, for failure to occur the resisting toe wedge must slide on the dump-foundation contact. The driving wedge(s) in the upper part of the dump will not usually extend a significant distance behind the crest.

- The shear strength of waste rock is a complex function of density, gradation and material type. Currently adopted design values are conservative.

- Long-term stability of dumps is dependent on continued drainage within and beneath dumps. Since the toe slope of most abandoned dumps is relatively flat the long-term performance is expected to be similar to that of natural talus slopes. We believe that conditions of humidity and temperature within the dumps fluctuate seasonally by relatively small amounts.

- Assessment of foundation conditions requires careful field inspection and selective materials testing. Selection of an appropriate factor of safety should be a function of the consequences and/or acceptability of

failure, the variability of field conditions, the degree of conservatism in selecting material parameters, and the design life of the dump configuration. The factor of safety should be defined with respect to the critical segment of the failure mechanism i.e. the toe region. This approach promotes a realistic assessment of stability and carries significant economic importance.

o Further Work

- Improvements in the routine documentation of waste dump behaviour are desirable. If well organized, we do not see that this additional documentation should cause a large extra workload. A standardized checklist for recording crest conditions, and material quality and rate should be developed. In the long-term mines may improve planning and efficiency by minimizing dump shut-downs. The geotechnical engineer at each site should maintain status plans which accurately show the toe line with respect to topographic features and areas to different foundation conditions.

- Further detailed observation of dump behaviour is desirable to complement monitor data. This will result in more complete sets of information and refinement of the trends documented in this report. Short-term programs of observation of selected dumps may be suitable as summer student tasks. Current pressures on mine personnel may hinder the systematic and unbiased study of dump behaviour that an independent observer would provide.

- Further research is required in both the field and laboratory to confirm that long-term degradation of dumped wate rock will not result in unstable conditions.

- Large scale testing of waste rock materials could be carried out by performing tilt tests as proposed by Barton and Kjaernsli (1981). The cost of such tests is expected to be moderate and could be conducted at several adjacent mine sites using the same box frame. The tilt tests would be complemented by analysis of particle size distribution, particle strength, and durability.

- Research into the application of accoustic emission monitoring at dump toes or crests may be worthwhile.

- Experimentation with automated methods of data collection from wireline extensometers may be advantageous if reliable and should be investigated.

- Refinement of work presented in this report on the runout mechanisms of failure debris is required both in the analysis of available data which has not been fully utilized herein, and in developing a more sophisticated analytical approach.

# 3.0 METHODOLOGY

## 3.1 Objectives

The overall objectives of the study were as follows:

(i) Review experience with past waste dumps: past failures, past behaviour (i.e. deformation) and past operating practice.

(ii) Analyze past events for patterns, correlations and unusual circumstances.

(iii) Use results of (i) and (ii) to:

- Review current practice for operating safety and efficiency;
- Refine understanding of mechanisms of deformation and failure, based on observation and technical analyses;
- Prepare improved predictive tools for prospective situations to be utilized by operators, designers and regulators.

## 3.2 Types of Available Information

### 3.2.1 Dump Performance Monitoring

Monitoring data on performance of waste dumps is obtained routinely at some operations and intermittently, as deemed necessary, at other sites. Types of performance data obtained include the following:

- o Measurement of dump crest deformation at regular time intervals using extensometers or survey techniques.
- o A limited amount of data on measured deformation behind the crest on the dump platform and on the dump face.
- o Observations and photographs of deformation on the platform and face of dumps.

### 3.2.2 Waste Dump Configurations

In reconstructing circumstances and events surrounding failure or high activity in a waste dump, we have placed considerable importance in defining the topographic and geometric situation at the time. Data defining geometric configurations includes:

- o dump face height;
- o slope in the toe region; and
- o profile of both the dump face and the underlying natural ground

Where applicable, information on the shape of topographic gulleys, which may have some effect on stability have also been defined.

### 3.2.3 Foundation Conditions

Available documentation of foundation soil and bedrock conditions has been obtained. The types of data reviewed include:

- o spatial distribution and thickness of different soil types;
- o soil index and shear strength results;
- o bedrock geology: lithology and structure; and
- o ground water flow regime.

Significant thicknesses of weak surficial organic materials or of ground water discharge have been specifically noted and highlighted.

### 3.2.4 Waste Rock

Available data collected on the waste rock generated at each mine includes the following:

- o strength and durability test results;
- o estimates of relative amounts of each material type consigned to dumps; and
- o estimates of rates of consignment to particular dumping areas.

### 3.2.5 Weather Conditions

Daily and monthly summaries of precipitation and temperature have been obtained where possible. Relative location of weather recording stations in relation to the dump sites is variable. Commonly weather stations are located in the valley floor, which may be as much as 500 m or 600 m below the mining area. The difference in conditions between dump sites and valley weather stations is variable and depends on the time of day and year and the type of weather system being experienced. Nevertheless, the available weather recordings are considered to be indicative of general weather systems in the area and provide an indication of significant trends and major weather changes. We are interested in the magnitude of major rain events such as summer storms, periods of long persistent wet weather, early unusually cold fall conditions which may permit significant depth of ground freezing and trends in spring thaw. The rise of the mean daily temperature or an abrupt rise in river flow during spring time has been used as an indicator of the onset of significant snow pack melt.

### 3.2.6 Failure Events

Documentation of past-failure events has included the following:

- o eye-witness accounts;
- o surveyed or estimated geometric configurations before and after failure; and
- o monitoring records preceding failure event.

## 3.3 Data Collection and Collation

The study process was initiated by holding telephone discussions with key geotechnical site personnel to review the type and extent of data collection required. Site visits were carried out during good weather conditions between mid-December, 1986 and early January 1987. The purposes of the site visits were:

- To meet mine personnel, explain objectives of the study, and answer any outstanding questions.
- To become familiar with the general layout of mine sites not recently worked on or visited.
- To observe winter operating conditions.
- To discuss in detail the objectives of the study, to reinforce the need for as complete a package of information as possible on particular events and monitoring periods.
- To review the quality and extent of available data to ensure that context of data supplied is not misinterpreted.
- To gain further appreciation of different techniques of operating and monitoring dumps common to specific sites; subsequently utilizing this familiarity to avoid inappropriate treatment and comparison of data. If practicable, data was retrieved during the visit. Otherwise, arrangements were made for mine personnel to carry out data collection.
- Where appropriate and practicable programs of additional data collection during the course of the study period were discussed and suggested.

Through discussion with site personnel we have attempted to identify specific site problems and characteristics, exploring reasons for apparent differences in operating conditions and behaviour between different mine sites. This process has also included discussion of different approaches to monitoring of waste dump behaviour and different strategies in controlling dump behaviour. Mine personnel have been

encouraged to express their experiences, views and concerns on waste dump performance and management.

Additional site visits conducted in January and February by Golder Associates during the course of other project work have provided an additional information on waste dump operation during winter conditions.

## 3.4 Data Collection

Based on a good level of familiarity with the waste dump operations, we have tended to focus on the most active/problematical waste dumps. In general we have looked at the more recent waste dumping history since it is better documented and more readily obtained. Current mine staff are more familiar with recent events and are better able to provide discerning interpretation of it.

The process of data collection has generally been iterative, involving review of data packages initially compiled by mine personnel, often leading to requests for additional explanatory information or data. Usually discussion of these additional requests has exposed less tangible, but nevertheless important, facets of operating experience. Where judged valid and practicable, these facets have been woven into our assessments and comparisons of waste dump behaviour. The objective has been to provide an undistorted, realistic comparison. Simultaneously, information held on file by Golder Associates has been retrieved, reviewed and merged with incoming data from the mines. This information includes design reports (with the results of field investigations including field notes and photographs), and performance reviews conducted on quarterly, annual or intermittent bases.

## 3.5 Data Collation

The objective has been to compile records containing sets of factual information relevant to an event or period of observation which are as complete as possible. The records may include comments based on judgment, experience or intuition, provided that there is a clear distinction between factual information and opinion. We have utilized a standardized format on a computer data base. The format seeks to ask a broad range of questions of both factual/objective and a subjective nature. The data base records are anonymous and are backed up by referenced data files which contain information such as monitoring records, topographic data, weather conditions, rate of dumping and quality of waste rock. Key parameters characterizing the data records have been tabulated in a consistent format using a computer spreadsheet. These tabulations are contained in Appendix B. Major deficiencies or uncertainties or contradictions evident in the data have been noted so as to avoid an inappropriate comparison or correlation of data.

## 3.6 Data Correlation and Characterization

In simple terms, the objectives are to seek relationships between causes and effects, and to rank and classify these relationships. In the past, general relationships between waste dump behaviour and contributing factors such as weather, material type, topography and rate of dumping have been apparent by observation of dump behaviour. However, the inter-relationship between these factors and their relative importance had not been well defined.

Tabulations of key parameters obtained from the collated data were sorted into various groupings using computer spreadsheet techniques. This process has led to the characterization of a set of dumping situations, each exhibiting a typical behaviour.

## 3.7 Peripheral Information Search

The objective of the peripheral information search was to obtain relevant published information regarding the observed behaviour of similar waste dumps, information on the engineering characteristics of similar materials, and relevant methods of analysis with respect to predicting deformation and state of stability.

### 3.7.1 Similar Materials

The purpose of the information search was to find data on the strength, deformation, and durability of loose rock matrices similar to materials being incorporated into the waste dumps. Information on materials containing particles with broadly similar shape, size, degree of packing or void ratio, and particulate strength was obtained. Most of the information available on the strength and deformation characteristics obtained from large scale testing of rock fill has been obtained for construction of major dams. Most of the available data published in recent years is for compact material.

### 3.7.2 Experience with Similar Waste Dumps

We have reviewed documented experience with mine waste rock piles located elsewhere in Canada, the United States, England, South Africa and Australia. This information is in the form of technical papers and in-house files held by Golder Associates.

Although broad similarities in the documented failure mechanisms exist, foundation slopes are generally flat. The influence of pore water pressures and their rate of dissipation, and the presence of specific weak horizons limit the applicability of 'foreign' experience to the B.C. Scene. With respect to the U.S. we have drawn on several

publications by and discussions with members of the Office of Surface Mining in the U.S.

### 3.7.3 Methods of Stability Analysis

The documentation of 'foreign' experience has included methods of analysis modified to simulate inferred mechanism of failure. The applicability of these methods to coal mine waste dumps in B.C. has been reviewed.

Experience with performance monitoring techniques tried elsewhere has been reviewed.

## 3.8 Technical Analyses

### 3.8.1 Stability Analysis

Data on the properties of mine waste rock obtained from the participating mines has been compared with similar material documented in the Peripheral Information Search. We have estimated ranges of material properties which control the shear strength and compressibility of mine waste rock. Ranges of shear strength parameters have been selected which are considered appropriate at different locations within waste dumps.

The sensitivity of the factor of safety against base sliding failure to variations in material strength at different locations within a dump has been calculated. Situations analyzed include the following:

- varying for waste rock angle of internal friction within possible realistic limits at different points within a dump;
- elevated phreatic surface;
- oversteepening of the toe and crest of the dump face;

- o apparent cohesion in waste rock with a high proportion of fine particle sizes;
- o hydrostatic pressure on the base of frozen foundation soil.
- o heavy truck loading near steep crest scarps; and
- o layer of relatively weak waste material inclined at 37 degrees, i.e., parallel to the dump face at some depth within the dump.

Methods of three-dimensional stability analysis have been reviewed and are discussed in Appendix E.

### 3.8.2 Runout Analysis

Data on the runout characteristics of post-failure events is included on the data base records in Appendix B. Information for selected events has been back analyzed to obtain a range of equivalent angles frictional resistance for base sliding of waste dump material under various circumstances. Work carried out includes a review of available literature and is included as a separate package in Appendix D.

# 4.0 DATA COLLECTION

The objective of Section 4.0 is to provide the reader with an appreciation of the extent of data studied and of its relative quality. Intepretation of the data will follow in Section 5.0.

## 4.1 Data Retrieval

As expected, the extent of records relating to waste dump behaviour has been found to vary from mine to mine. Records of older events tend to be less complete. In the data retrieval process, we have striven to obtain as complete a package of data for a particular event as possible, with the objective of comparing only genuinely similar sets of circumstances or alternatively, of identifying as clearly as possible, the principal reasons for documented differences in behaviour.

Key parameters entered into the database have been summarized in spreadsheet format, permitting sorting of groups of data by any desired combination of parameters. On the basis of spreadsheet tabulation, various tables, bar charts and graphs have been generated which illustrate linkages and relationships between various parameters.

## 4.2 Classification of Data

This section describes the various categories of data collected and collated and any assumptions during this process, and comments on quality of data.

4.2.1 Type of Dump Behaviour

The following sections summarize the types of failures observed and documented. The database records of failure events in Appendix B have been sub-divided into these categories.

1. Major Dump Failures

Major failures involving large volumes of material: All of these failures are initiated in the toe region of the dump. Where available, numerical data on the extent of major failures has been recorded on the database i.e. runout distance, dimensions and volume involved, and observed or inferred velocity. Some observations of oversteepening in the toe region of the dump face have been reported and recorded on the database. Available eye-witness account key phrases and inferences for other non-observed events have been recorded. In some cases, estimates of the failure surface profile have been made.

2. Sliver Failures Involving Relatively Small Mass

This type of failure appears to be initiated either in the toe or crest region. The primary distinction of a sliver failure as opposed to a larger mass failure is that the failure mechanism appears to be relatively shallow. Also, the resulting quasi-stable deformed mass may not travel large distances beyond the original toe position, and characteristically forms a series of slump steps near the crest as opposed to a single, large, steep scarp.

3. Creep Failures

Large displacements are experienced but the rate of displacement does not increase significantly. This type of mechanism may be associated with spreading failure of weak foundation material.

4. Failure of Inactive Dumps

Failures of several inactive dumps have been experienced. These failures appear to be related to one of the following:

- o increase in pore water pressure acting at the dump foundation contact;

- o reduced strength of waste rock due to thaw of buried snow or increasing moisture content; and
- o failure of highly stressed waste rock particle contacts along steeply inclined dump-foundation contact surfaces.

4.2.2 Periods of Increased Crest Deformation

In reviewing available records of dump behaviour, we have been interested in not only major dump failures, but also in periods of increased dump activity, and also in "normal" levels of activity.

Periods of high activity, i.e., large rates of deformation have been experienced at most of the high dumps located on relatively steep terrain. Relatively high rates of movement will be experienced, usually as a result of relatively high rates of dumping. However, upon cessation of dumping in response to the measured or observed high rates, the rate of deformation will either increase to failure or decrease permitting resumption of dumping activity. Records of these periods with high rates of movement have been collected and sorted according to contributing conditions including geometry, weather, material type, as documented for failure events. If available, visual observations have been taken into account. In several situations there has been an opportunity to measure the direction of movement in the crest region, both in the vertical and horizontal planes.

Visual observations of deformation of the faces of waste dumps have also been examined. The following types of features have been observed:

- o Bulges and horizontal patterns of strain in the upper part of the dump.
- o The progressive translation of bulges down the face.
- o Over-steepening of the toe region.

- o Development of melt zones in the middle third of dumps during winter.

The following types of data have been examined:

1. Background rates of crest movements preceding periods of increased activity.

2. The characteristics of acceleration to a peak during active periods.

3. The relationship of active periods to various contributing factors, i.e., weather, material, type, rate of dumping, foundation conditions and geometry.

4. Movement vectors for crest and face deformation for several mine sites.

### 4.2.3 Performance Monitoring Data

The most common indicators of the behaviour of waste dumps are qualitative assessments based on inspection of crack development, and displacement monitoring at the crest. Monitoring of deformations visible and measurable at the crest of waste dumps has been the main method of assessing the relative state of stability of waste dumps.

Methods for monitoring activity at other locations within the waste dumps have not commonly been utilized for practical reasons. That is, measurement of deformation and pore pressure in the toe region of the waste dump, both within the dump and in the foundation materials, has not commonly been carried out. Such installations have been viewed as being either impractical, costly and of insufficient benefit in comparison to crest monitoring methods. Similarly, installation of instrumentation within the body of waste dumps has not been seen as having immediate benefit to efficient operation of waste dumps. Table 4.1 summarizes the various types of monitoring which have been carried out at coal mine waste dumps in British Columbia.

Table 4.2 shows the type of monitoring device used at each of the operations studied. Wire line extensometers are currently used at four operations which appear to possess the least stable dumps.

Tables 4.1 and 4.2 summarize the type of monitoring data normally collected and available for past events. Additional observations in the form of photographs have been collected by Golder Associates over the last 15 years and also by some of the operating companies. Appendix A contains a selection of relevant photographs which illustrate key phenomena.

4.2.4 Geometry

Typically, the waste dumps are formed on the side slopes of glaciated valleys which have a characteristic U-shape profile. Topographic relief from these valleys is in the general range of 300 m to 600 m. Fall line slopes may be in the range of 30° to 40° at the higher elevations, gradually flattening to less than 10° at the lower elevations. Wherever possible waste material mined above and between coal seams, is disposed of by end-dumping at an elevation close to the mining level. Aside from stability considerations, the principal planning objective is to minimize the haulage distance, and wherever possible to avoid dumping over potential future coal reserves. Thus, waste dumps in British Columbia would fall into the "sidehill" classification. Some dumps are constructed as a series of benches such that the loes of the upper benches are supported at all times on stable benches of lower elevations.

Types of topographic setting for mine waste spoils in British Columbia can be divided into three general types as shown on Figure 4.1. Six of the eight operations involved in the study have sidehill fills. Three operations have some dumps in which can be construed as the wrap-

around type. One operation has a valley fill for in which the direction of crest advance is parallel to the axis of the drainage course. Three of the operations have some backfilling of defunct mine pit areas. Figure 4.1 shows the principal topographic settings. Sidehill fills are the principal type under consideration in this study. Wrap-around fills are seen as a variation on the sidehill fill concept. Side hill fills can be subdivided into different crest shapes as shown on Figure 4.2.

Figure 4.3 shows different profiles of natural topography. Type A is a concave slope flattening with decreasing elevation. Type B is a convex slope steepening with decreasing elevation, and Type C is a uniformly sloping segment of topography. A typical example of Type C is where steep topography exists at a slope which is approximately parallel to the face of the dump.

Crests of sidehill fills might be advanced in directions ranging from parallel to perpendicular to natural ground contour. Figure 4.4 shows the effect on dump surface profile of varying direction of crest advance. The variation on the "sidehill" fill is the wrap-around dump which is typically a relatively narrow dump platform that is advanced along contour over the face of an existing dump. Typically the slope of the foundation that supports the wrap-around is somewhat flatter than the slope of the pre-existing dump. An exception to this would be the configuration shown on Figure 4.5(b) where a new dump platform has been constructed in an artificial gulley formed by steep topography and an existing dump face. There have been relatively few "valley fills" in which the dump crest is advanced parallel to the principal drainage direction. The toe slope beneath fills in major valleys would be relatively flat. We are unaware of any major problems with valley fills. As a result little mention of them is made in this report.

Figure 4.5 shows two types of dump development which have been carried out in natural and artificial topographic v-shaped gullies. The shape of the gully can be defined by the angle of intersection between contour lines along the line of the base of the gully. The extent of wedging action in a gully will be dependent on this angle and also upon the inclination of slopes forming the gully.

The importance of controlling direction of crest advance in relation to natural topography is discussed in a subsequent section. The quality of topographic information is generally adequate, with contour intervals in the general range of 2 m to 10 m. Minor departures from reality, commonly seen in computer-generated topographic maps, are not of major consequence. In selecting topographic profiles we have frequently chosen a curved line in plan view. This would be intended to reflect the anticipated direction of movement that a potential failure mechanism might take. The use of straight line sections, which do not follow drainage courses, can give the misleading impression that toe slopes are flattening.

### 4.2.5 Foundation Conditions

It is reasonable to state that the general range of foundation conditions are broadly similar for each of the mine sites considered in the study.

Sedimentary bedrock strata comprise varying proportions of sandstone, siltstone, mudstone, shale, and thin, uneconomic coal seams. Generally, bedrock strata are exposed or exist close to ground surface on the steep upper slopes of waste dump areas. In general, bedrock strata dips relatively gently (i.e., at angles of less than about 30°).

It is noted that the mudstones and shales are free of significant amounts of bentonitic minerals but are susceptible to relatively rapid slaking when exposed.

Colluvial soils derived from these bedrock strata are usually found mantling the slopes below to depths of up to about 1 m. The colluvial soils comprise varying proportions of silt, sand, gravel and cobble-sized particles. Larger particles are usually quite angular and the soil matrix is usually in a loose to compact state. Figure 4.6(a) shows a collation of typical grain-size data for colluvial soils. The results of shear-strength testing on reconstituted samples of colluvial soil generally results in a drained angle of internal friction in the range of 33° to 38°.

It is common to encounter significant thicknesses of glacial till underlying colluvial soils on the middle and lower slopes of the waste dump foundation areas. Figure 4.6(b) shows a typical range of gradations for the glacial till materials, and it can be seen that the percentage of fine grain silt sized material can range between about 20% and 50%. Generally, the till materials are well graded, the consistency of the till materials ranges from hard, very densely-packed to a softened, often reworked (or slumped) relatively loose consistency. Figure 4.7 shows shear strength test data for both glacial till and colluvial soils.

In general, waste dumps in British Columbia have not been founded on areas of major ground water discharge. However, where it is evident from reports of waste dump failures, that discharge seepage in the foundation area is evident, it has been noted in the database record.

Similarly, waste dumps have not generally been constructed over areas mantled with significant thicknesses of weak organic materials.

However, where a significant thickness of organic material is believed to be a factor in dump failure, it has been documented. It should noted that removal of forest cover, grubbing, and removal of topsoil is not generally carried out. In fact, it has been found that the process of removal of topsoil and vegetative matter can result in significant disturbance of foundation soils, resulting in a surficial layer of loose saturated fine-grained material. Silty glacial till soils, when disturbed and loosened, tend to exhibit an affinity for any available moisture.

### 4.2.6 Climatic Conditions

Generally, climatic conditions are similar at each of the mine sites involved in the study. This is illustrated in Figure 4.8.

Figure 4.8 shows a monthly summary of weather conditions during 1985 and 1986 for available weather stations. Seasonal fluctuations in temperature and precipitation appear to vary quite uniformly. The mean monthly temperature for both the northeast and southeast coal block regions is contained within a 4 to 5 degree band. It is apparent that spring thaw is initiated during March and April. Winter freeze up appears to occur relatively rapidly in November. It is noted that the unusually rapid fall in mean temperature in the late fall 1985 is thought to have been responsible for one dump failure experienced in 1986 due to frozen ground.

The total monthly precipitation also appears to vary quite consistently. For most months the northeast region receives somewhat less precipitation than the southeast. There is much less variation in the dryer months than in the wet ones. This probably reflects the greater spatial variability of storm events in mountainous regions. The total monthly precipitation appears to fluctuate, with highs and lows

alternating with approximately a monthly frequency. In 1985 the months of August, September and October were relatively wet, presumably due to summer storms. By contrast the same period in 1986 was significantly dryer.

It should be noted that most of the weather stations are located at elevations which are significantly lower than the dump areas. The lower bound of temperatures shown on Figure 4.8 probably reflect the variation in temperature at the waste dump sites. Thus spring snow melt could be expected to be properly underway in the latter part of March and in April.

Significant weather events which appear to have some relationship with waste dump behaviour are:

- Spring thaw during April and May, as indicated by temperature and river flow.
- Summer storms during June, July, August and September. Magnitudes in excess of 30 mm/day appear to have some relationship to increased incidence of failure or rate of movement. We have looked for possible relationships between major precipitation events in periods of one to two weeks preceding a dump failure or period of marked increase in rate of movement.
- In one instance, the occurrence of an unusually cold period of weather in mid-fall, in the absence of a significant cover of snow (as insulation), appears to have resulted in failure the following summer due to an increase in pore water pressure beneath the frozen ground surface along the dump foundation contact.

River flow data have been used to give an indication of the onset of spring thaw, accepting that there may be some small differences in the timing of groundwater rise due to spring snow melt and of river flow increase. However, the four or five fold increase in river flow which occurs over a period of several days appears to be a reasonable and reliable indicator that melt of the snow pack is making available this

very significant source of moisture for infiltration into mountain slopes above waste dump areas.

4.2.7 Waste Rock Quality

Factors which affect the quality of waste rock in a particular dump include:

- Geology of the mining area, i.e., rock type and the joint and bedding spacing. Characteristics of the material type which can be indexed are grain size, that is sandstone, siltstone or mudstone; strength quantified by such tests as unconfined compression and point loading; and durability quantified by tests such as slake durability, Los Angeles abrasion, and sulfate soundness testing. An additional valuable tool in assessing durability is visual observation of outcrops and of fragments produced during mining.

- Relative proportions of different types of rock controlled by mine geology and also by the mining sequence and direction. In addition, material may be consigned to a particular dump from multiple sources. The relative proportions of material coming from different sources is clearly a day-to-day operational variable.

- Other factors which may influence the quality of waste rock reaching a particular waste dump are blasting practice and degree of rehandling. Overall, it is felt that blasting is of secondary importance to in-situ fracture spacing in determining material size. The proportion of severely shattered particles due to blasting is thought to be relatively small.

- Although not well documented, it has been our experience that, in some cases, periods of dumping relatively fine grain material are attributed to rehandling. If available, information has been incorporated as an identifiable factor in waste dump performance in the case history records.

Table 4.3 shows the range of values of unconfined compressive strength for various material types from different mine sites. The information is not necessarily statistically representative with respect to the overall quality of waste rock consigned to any particular waste dump. The data provides a general comparison of conditions at different

sites. A more reliable means of assessing the effect of dump material quality on dump performance would be to record the general quality of material entering a waste dump on a regular basis.

### 4.2.8 Rate of Dumping

Data on the rate of dumping of waste rock have not been consistently collated for the mine sites. However, reasonably detailed data is available for two of the mine sites which have consistently experienced the most activity. For these two cases we have used rate of dumping per meter of active crest length. The fact that the two mine sites with the most active dumps have taken the most interest in rate of dumping as a controlling parameter confirms its importance. Even in the two relatively well documented cases, it has not always been clear that a given documented rate of consignment applies consistently to the same segment of crest length. Overall we consider that the data has given a good, if not precise, indication of dumping activity relative to given documentation of dump performance in the form of crest deformation records.

## 4.3 Ongoing Data Collection

### 4.3.1 Dump Behaviour Observation

Mine geotechnical personnel were requested to carry out observation of waste dump behaviour, i.e., surface deformation for the period of study during the months of January, February and March, 1987. Regular measurements or photographic records of face profiles of selected dumps were requested. Documentation of the condition of the dumps including the degree of crest deformation, the type of material being consigned to the dump, and any other possible contributing factors was requested. Mine geotechnical engineers were requested to summarize their general

observations, and operational problems and strategies employed to deal with increases in waste dump deformation.

### 4.3.2 Material Sampling

Random sampling of typical waste dump material has been carried out at several mine sites. At Site C the density of material in the crest region in site was determined. Date from other sites on the gradation of waste rock were obtained. The results of this testing are discussed in Section 7.1.

### 4.3.3 Operational Methods and Problems

Operating procedures for waste dump development at individual mine site have been collated and reviewed. This information tends to be more detailed for mine sites where movements are relatively high. Combined with the summaries provided by mine geotechnical engineers we have been able to compare the operating conditions and problems for different mine dumping situations.

# 5.0 DATA ANALYSIS AND CORRELATION

## 5.1 Seasonal Trends

The principal purpose of Figure 4.8 is to illustrate the general similarity in weather conditions at each of the mine sites considered in the study. The following sections discuss in detail the behaviour of dumps for which moderately complete data is available over a period of one or more years.

### 5.1.1 Site A

Figures 5.1 and 5.2 show the variation of crest displacement rate for seven dumps at mine Site A during 1986. The dumps have been divided into two groups according to geographical location.

#### Site A - Group A Dumps

Slopes in the toe region of Group A dumps range between 14 and 34°, with dump face heights in the range of 160 to 190 m. From January through April measured rate of crest displacement was generally less than or equal to 0.5 m/day. This dump performance corresponded to mean temperatures generally less than or equal to 0°C and rates of dumping of between 100 and 150 bcm/m/day. In early May, the mean temperature rose consistently above 0°C. At the same time creek flows began to rise sharply, as evidenced by a significant rise in the level of a nearby sediment control pond. This occurrence is considered to reflect the onset of major melt of the winter snow pack. Group A dumps appear to have shown a mild response to the onset of spring thaw. Rain storm events are also shown on Figure 5.1. There appears to be some relationship between rainfall events exceeding about 25 mm/day and quasi uniform response exhibited by most or all of the dump crest monitors monitors.

In early June a major failure occurred in one of the dumps as shown on Figure 5.1. It is possible that a major storm in mid June contributed to the accelerating trend which culminated in the failure at the end of June. Between July and November various dumps exhibited crest displacements in excess of 0.7 m/day for relatively short periods. There may be some relationship between these high rate periods and peak rainfall events in the range 15 mm to 25 mm as shown on Figure 5.1.

Site A - Group B Dumps

Group B dumps range in height between 200 and 225 m. In general, seasonal behaviour of Group B dumps is similar to that of Group A dumps. Response to spring thaw is more marked for Group B than for Group A dumps. Major failure occurred at one of the Group B dumps in mid June, and this appears to be linked to a major storm event several days prior. Between July and October it is apparent that factors other than storm events resulted in peak rates of crest displacements.

5.1.2 Site B Dumps

Figures 5.3 through 5.6 show the behaviour of a series of dump platforms constructed over a four year period. The initial stage of each dump platform was constructed by advancing the toe into an artificial gully formed on one side by steep natural topography and on the other by the face of the preceding platform. This configuration is illustrated on Figure 4-5(b). As the toe of a new platform advanced the slope decreased, height increased and wedging in the artificial gully diminished.

Rates of crest displacement were monitored using a two stand wireline extensometer. Figures 5.3 to 5.6 show smoothed trends in rate of crest displacement with weather conditions and estimated dump height and toe slope.

Average rates of crest displacement were generally less than 0.3 to 0.4 m/day and showed imperceptible response to major storm events. With the exception of a sliver failure event in the spring of year three, response to spring thaw conditions was also minimal.

We attribute the generally low rates of crest displacement to a combination of coarse waste rock and good control of direction of crest advance during initial stages of develoment.

Periods of increased rate of crest displacement are probably the result of consistently poor rock quality.

5.1.3 Site C

Figures 5.7 and 5.8 show seasonal fluctuations in crest displacement rates during 1986 for a major dump with a toe slope of about 24° and a face height of 250 m approximately. The records of three monitors spaced approximately 80 to 100 m apart on the dump crest are shown on Figure 5.7.

Based on mean temperature values spring thaw is expected to have begun in the middle part of May. Each of the three monitors appears to have responded to spring thaw. However, it is noted this response may be affected by other factors including consignment of poor quality material to the dump and a marked increase in the rate of dumping in an adjacent dump area.

As the mean temperature fell consistently below 0°C in November measured rates of crest displacement appear to have decreased. However, it is also noted that at this time the rate of dumping was also relatively low.

Figure 5.8 shows the variation in crest displacement measured during 1986 for a new dump with the toe founded on slopes in excess of 30°. The monitors on this dump are referred as Group B. As with the Group A monitors shown on Figure 5.7 there appears to have been some response to the onset of spring thaw in late May and early June. Obvious correlations with storm rainfalls are not apparent with the exception of storm events in mid February and late August.

Figure 5.9 illustrates seasonal variations in movements observed on the surface of an inactive dumping area over a three year period. In the absence of any active consignment of waste material to the dump the effect on the rate of crest displacement caused by seasonal fluctuations

in precipitation is much more noticeable than for active dumps. However, it is noted that the rate of displacement is approximately one order of magnitude less than measured at the active dumping areas represented on Figures 5.7 and 5.8. Based on the assumption that spring thaw begins to occur between April and May in each year it appears that spring thaw has significantly less effect than major precipitation events. The rate of movement appears to increase by the largest amount as a result of high frequency of precipitation events during August and September of each year.

It is clear that intensity of rainfall can have a significant effect on rate of deformation on the face of a waste dump. However, other factors such as rate of dumping and consignment of fine grained material are considered to have a significantly greater effect.

Figures 5.1 through 5.9 show the occurrence of both failure events and periods of high rate of crest displacement versus month of the year. Peak periods from April to June and from August to September are evident. These two periods of peak activity appear to correspond respectively to spring thaw coupled with summer storm activity, and storms in the late summer or early fall.

## 5.2 Effects of Geometric Configuration

The range of different types of geometric configuration studied was discussed in Section 4.3.1. Figures 4.2 through 4.5 illustrated different plan and profile configurations which apply to the dumps covered in this study. Where information is available each waste dump situation studied has been classified in a consistent manner. The following sections describe correlations noted between waste dump behaviour and various classifications of geometric configuration.

### 5.2.1 Profile Shape

Table 5.1 shows that 65% of the failure events studied occurred with Type A geometric profiles, i.e., topographic slope decreasing with decreasing elevation. The geometric profiles for the other 35% of failure events studied were more complicated. Failures associated with a Type B profile generally occurred as the slope in the toe region gradually steepened above about 25°.

### 5.2.2 Toe Slope

FREQUENCY OF FAILURE

The average topographic slope of the foundation in the toe region has been estimated for each failure event identified. Figure 5.10 shows the frequency of failure occurrence against interval of slope inclination. Eighty per cent of all documented failure events have occurred on slopes inclined at angles in excess of 24°. The remaining twenty per cent of slope failures have occured for slopes inclined between 10° and 24°. Table 5.2 ranks the identified factors affecting the failures. The ranking has been divided into two categories for 'steeper' slopes exceeding 24°, and 'flatter' slopes inclined at less than 24°. It is significant that steepening of the natural topographic slope in the toe region is identified as affecting about 11% of steep slope events and about thirty-three percent of events on flatter slopes.

The frequency of occurrence of events involving accelerating crest movements to rates exceeding one m/day are also shown on Figure 5.10. The distribution is similar to that for failure events. Figure 5.10 demonstrates that the majority of failure events correspond to foundation toe slopes steeper than 24°.

TIME TO FAILURE

Figures 5.11, 5.12 and 5.13 show plots of the measured crest rate of displacement, plotted against time before failure for very steep slopes in excess of 30°, for moderately steep slopes between 20° and 30°, and for relatively flat slopes at less than 20°. It is noted that all of the crest displacement records for failures have been obtained using wire-line monitors either with one or two stands. Dump heights vary between about 100 m and 300 m. Factors that were probably significant in initiating the failure event are tabulated on the diagrams. It is evident that dumping activity ceased at least one hundred hours prior to most of the failure events. The curves shown on Figures 5.11 through 5.13 show a broad range of acceleration characteristics, reflecting a corresponding broad range of conditions.

An important parameter is the amount of time between the measured rate of crest displacement exceeding a certain value and the time at which dump failure occurs. Figures 5.11 through 5.13 indicate that in all cases shown the time before failure after a rate of 0.5 m/day is exceeded is generally in excess of one hundred hours.

Figures 5.14 and 5.15 show time before failure after rates of 1 m/day and 2 m/day have been exceeded, plotted against toe slope. The distribution of plotted points on Figure 5.14 indicates that for dumps on slopes of less than 30° the time to failure subsequent to a crest displacement rate of 1 m/day is at least twenty hours. Distribution of points plotted on Figure 5.14 indicates that with decreasing toe slope the time to failure increases. Similarly, in Figure 5.15 the distribution of points shows increasing time to failure past 2 m/day for decreasing toe slope.

Figure 5.16 shows time to failure past a selected critical rate plotted against that critical rate for the range 0.5 to 2 m/day. The range and average values for the available data points are shown. Since the number of data points is limited it should be remembered that the average values are not necessarily statistically representative. However, the trend indicated by the lower bound of plotted ranges indicates worst case conditions.

The following general conclusions can be reached from examination of Figures 5.14 through 5.16:

- For toe slopes steeper than 30°, the worst case recorded indicates a minimum time to failure of about 4 hours. This is believed to be an extreme situation corresponding to poor waste rock combined with excess pore water pressure.
- Considering average values of time to failure for steep slopes in excess of about 30 degrees, for a selected critical crest displacement rate in the range 1 to 1.5 m/day the time to failure is in the range thirty five to fifty hours.
- Considering the lower bound for toe slopes of less than 30°, with a selected critical crest displacement rate in the range 1 to 1.5 m/day the time to failure is in the range twenty four to forty hours.

ACCELERATION

Figure 5.17 shows rate of displacement plotted against the logarithm of time to failure. This method of plotting results in an approximately linear trends. The slope of the linear segments gives an indication of acceleration. The slopes of each failure plot, for rates of displacement exceeding 1 m/day, have been calculated and are expressed as an 'acceleration' with units of m/day per day. This parameter A has been plotted against toe slope on Figure 5.18. Figure 5.18 does not show a clear correlation between 'acceleration' and toe slope. It appears that a relatively small number of points exhibit

markedly higher acceleration trends than the majority of the cases plotted.

The overall conclusion appears to be that in most cases a significant margin of time exists between the time at which the crest displacement rates exceed 1 to 2 m/day and the time of failure. In general the acceleration exhibited does not appear to be a direct function of toe slope.

In most cases the dumps being monitored prior to a failure event had been actively dumped upon within a week of the failure event. Most of the dumps shown on Figures 5.11 through 5.13 had been shut down at least one hundred hours prior to the failure event. We infer that visual indications of distress, i.e., cracks and the development of step-like profiles in the vicinity of the crest either prevented further practical dumping or other visual indications gave dump operation personnel warning of impending failure.

NON-FAILURE EVENTS

Periods of increased rate of crest displacement which exhibit an accelerating trend but which do not lead to a failure event have also been studied. Figures 5.19 through 5.21 show rate of displacement against the time interval before a peak value was reached. As for failure events the plots have been divided into categories of low, medium and steep toe slopes. For the steeper slopes characterized on Figure 5.19 the accelerating trends are similar to those drawn on Figure 5.11 for failure events. However, the peak rates of displacement do not exceed 4 m/day. Comparing Figure 5.20 with Figure 5.12, the general similarity in acceleration trends is apparent, although the medium slopes depicted in 5.12 are several degrees higher on average. Also, it may be significant that in general the high rate events, which have not

resulted in failure, apparently are not linked to spring snow melt or to heavy rainfall events immediately prior. This may be further evidence that links the occurrence of failure events to spring snow melt. There do not appear to be any clear distinctions between the acceleration trends for low and moderate toe slope. However, the events for lower toe slope appear to be more strongly related to spring snow melt, or to conditions or poor rock quality.

Figure 5.22 shows semi-log plot versions of Figures 5.19 through 5.21. The indicated values of 'acceleration' are almost all less than 4 m/day per day, and places the acceleration trends consistently in the low range of accelerations noted for major failure events on Figure 5.18.

BACKGROUND RATES

Figure 5.23 shows normal or background rates of crest displacement taken over a 1-2 week period. The rate of dumping appears to be a more dominant factor affecting displacement rate than toe slope or dump height. The upper bound of data points may indicate a trend for increased rate of displacement with increased toe slope.

### 5.2.3 Dump Height

FREQUENCY OF EVENTS

Figure 5.24 shows the frequency of both failure events and high crest rate events against dump height. 45 percent of the failure events were for dump heights in excess of 200 m. The number of failure events for slopes of less than 100 m height is relatively small at 6 percent. This conclusion may reflect a bias in the group of dumps studied towards more recent and better documented dumping scenarios. We cannot conclude that dumps of lower height are always more stable.

ACCELERATION

Figure 5.25 shows the slope, A, of the semi-log plots of crest displacement versus time to failure plotted against dump height. The parameter A gives an indication of acceleration of crest displacement. The scatter of points shown on Figure 5.25 indicates that parameter A is not a direct function of dump height.

The scatter in points shown on Figure 5.25 may reflect the variable time between the last reading and the failure event itself. In fact, definition of the time of failure may be uncertain plus or minus six hours or more. We consider that the scatter of data points also indicates the complex interaction of factors affecting rates of deformation.

BACKGROUND DISPLACEMENT RATE

Background rate of crest displacement plotted against dump face height is shown on Figure 5.26. High rate of dumping appear to be more significant than dump height. Also, the lowest and highest toe slopes appear to result in the highest background rates.

PEAK DISPLACEMENT RATES

Plots of peak rate against toe slope and dump height are shown on Figures 5.27 and 5.28. These plots are for failure events, and are the last reading obtained prior to the failure event.

Figure 5.27 shows peak rate plotted against dump height for non-failure events. The density of data points is significantly higher for

the 200 m to 300 m height interval, and there appears to be some relationship between higher peak rates and higher dumps. Figure 5.28 shows peak rate plotted against toe slope for all dumps in the height range 190 to 260 m. Factors attributed to the high rates are also shown on Figure 5.28 and may indicate that spring snowmelt and high rate of dumping are significant factors resulting in higher peak rates for non-failure events. More complete sets of data are required to verify this inference.

SITE B EXAMPLE

Figure 5.29 shows background rates plotted against height and toe slope for Site B dumping platforms. Conflicting trends are evident and may be due to deviations from the optimum crest orientation for maximum benefit of three-dimensional wedging action.

Figure 5.30 shows rate of vertical settlement in metres per day against dump height for adjacent segments of dump which have similar toe slope but variable face height. Measurements were taken on one day only and the apparent approximately linear relationship may be coincidental. However, since crest displacements represent the integral of strains over dump height, it is reasonable to expect that total crest displacement, as well as rate of crest displacement for high dumps would be greater than for low dumps. In the absence of other data obtained under controlled conditions, this data is worth considering although it should be subject to confirmation in subsequent studies. The rate of dumping was estimated to be approximately 250 bcms per metre per day.

LOW DUMP FACE HEIGHT

By contrast, significantly lower background and peak crest displacement rates have been observed for sites G and F, for dumps in the

range of height 20 to 150 m. Figure 5.31 shows peak rates of crest displacement plotted against dump height obtained from available data for sites G and F. Most of the points shown on Figure 5.31 exhibiting rates of less than 0.1 m/day correspond to inactive dumping situations. It appears that a 'creep' rate for dumps with heights in the range of 0 to 150 m is of the order of 5 to 100 mm per day.

Background rates range from 2 to 40 mm/day for dumps with toe slopes in the range of 0 up to 30 degrees. Peak rates experienced for these dumps range from about 3 mm a day up to 25 m/day. The highest rate appears to constitute a creep failure due to foundation spreading. Subsequent construction of a wrap-around dump produced peak rates in this case to the range of 10 to 100 mm/day. Several measurements for inactive dumps with heights of up to 100 m exhibit similar crest displacement rates ranging from 10 to 100 mm/day.

COMPARISON OF LOW AND HIGH DUMPS

Comparing figures 5.27 and 5.31 there appears to be a difference between the peak rates for non-failure events and the creep rates shown on Figure 5.31 of approximately one order of magnitude.

It should be noted that there are more data available for non-failure events in the 150 to 300 m high range for active dumps as shown on Figure 5.27 than for relatively inactive dumps with face heights in the region of 0 to 150 m as shown on Figure 5.31. It would be a mistake to combine the data shown on figures 5.27 and 5.31 to infer trends for active dumps over the full range of dump heights of 0 to 300 m, since the data for inactive dumps reflects longer term creep and consolidation rather than shear strain due to recently placed material.

### 5.2.4 Three-dimensional Factors

This section discusses situations where the topographic lay-out results in a three-dimensional wedging action which increases the factor of safety against basal sliding of waste dumps. Two examples of waste dumps which have been designed taking into account this three-dimensional factor and which in fact have performed satisfactorily are discussed in this section.

Most of the dump platforms at Site B which have been characterized on Figures 5.3 through 5.6 have taken advantage of three-dimensional wedging action in the early stages of dump development. Figure 4.5(b) shows a plan view illustration of dumping strategy employed at Site B. The natural topographic slopes supporting the early stages of dump development are generally in excess of 30 degrees. The indicated factor of safety against base sliding of a planar sidefill placed on a slope inclined at in excess of 30 degrees could be expected to be at or close to unity, assuming realistic foundation shear strength parameters for the foundation soils in the region. The absence of failure events in the early stages of each dump platform development can be taken as an indication that three-dimensional wedging was successfully utilized.

Figure 4.5(a) illustrates a dumping configuration which has utilized a prominent v-shaped gulley. During the initial stages of dump development, the toe of the dump would have been supported on slopes in the range 25 to 30 degrees. Relatively low rates of crest displacement occurred with an absence of detectable face deformation. The favourable performance of the dump appears to demonstrate the value of taking into account the three-dimensional factor.

## 5.3 Effects of Foundation Conditions

As discussed in Section 4.3.2, the foundation soils consisting of colluvium and glacial till are generally similar or common to each of the mine sites studied. Figures 5.32 shows the distribution of factors identified as being significant or predominant causes responsible for failure events as well as periods of high rates of crest deformation. Overall, it is evident that relatively few major (reported) failure events are related to weak foundation conditions. Table 5.3 summarizes failure events associated with identified weak foundation conditions. Foundation conditions affecting or related to dump failures can be divided into three categories, as shown on Table 5.3:

- o Saturated, unconsolidated, surficial foundation soils -
  - Three major failures appear to be related to the presence of saturated, surficial soils. Two of the three identified events have been classified as relatively slow moving or creep failures. It has been our experience that numerous other minor cases of localized toe spreading have occurred at most of the mine sites at some time.
- o Foundation seepage
  - The three cases cited, which appear to be compounded by poorly drained, fine grained waste material occurred at the same site. In this case, the origin of seepage is not clear and may be the result of mis-directed pit drainage.
  - Two events appear to be related to the development of seepage pressure beneath a veneer of frozen surficial soil. In one case, the failure occurred several weeks after dumping activity ceased. Although monitoring records are not available for the period immediately prior to failure, significant deformations had occurred well back from the crest during the interval that dumping was inactive. In the case of event No. 149, it is felt that an unusually cold period in the fall of the preceding year resulted in the dump advancing over a significant area of frozen surficial soil.

o Previous failure debris -

- Two failure events have been identified which may be related to the presence of debris from previous slides which had occurred when the toe region of the dump was founded on steeper topography at higher elevation. Other factors such as seepage discharge or the presence of significant quantities of fine-grained waste rock have not been identified by the parties investigating these events. The possibility that significant quantities of snow had been incorporated within the waste material, either in intact dump or in the preceding failure debris advance, should also be considered.

The toe slopes for most of the events shown on Table 5.3 are less than 20 degrees. Assuming a normal range of shear strength parameters for foundation soils, without significant excess porewater pressure present, the factors of safety against base sliding along the dump foundation contact in the toe region of the dump could be expected to be 1.3 or more. Thus, significant excess pore water pressures must have developed along the failure surface.

## 5.4 Effect of Waste Rock Quality

As discussed in Section 4.3.7, waste rock at each of the mine sites is derived from rock strata of similar geological origin. However, variations in the degree of weathering and also tectonic shearing and faulting can result in a fairly broad range of particle size distribution for waste rock, and in the strength and durability of individual particles.

Figure 5.32 shows that a significant number of failure events have been identified as being related to a predominance of fine-grained material present in the dump.

Table 4.3 shows estimates of relative proportions of sandstone, siltstone, mudstone or shale considered to be representative, overall,

for waste materials. The highest rock quality corresponds to proportions of between 80 and 90 per cent of what can be classified as durable sandstones and siltstones. In summary, the material consigned to sites B, C, E and F can be classified as predominantly durable siltstone or sandstone with 70 to 90 per cent of those materials present. At sites A, D and G the proportions of finer-grained, less durable mudstones and shales appear to be significantly greater. The failure events shown on Figure 5.32 as being related to waste rock quality correspond to this latter grouping of mine sites generating proportionately larger amounts of less durable materials. The quantity of data on the quality of material being consigned to dumps prior to failure events is limited. The events identified on Figure 5.32 generally represent qualitative opinions expressed by various personnel before or after failure events.

Periods during which disposal of poor material is clearly identified, are shown on Figures 5.1 through 5.8. For Site B depicted on Figures 5.3 through 5.6, we consider that predominantly durable material has been consigned to the dump. In this case, less durable materials have been diverted away to stable dumps on relatively flat terrain.

It is quite probable that the presence of significant zones or layers of finer-grained, less durable material has contributed to either failure events or periods of high crest deformation which have not been reported as being linked to the presence of fine material. As part of recommendations for future study, and also for improved routine documentation of dump performance, more effort and emphasis should be placed on reporting material type and quality.

We consider that a significant number of failure events which have been attributed primarily to such factors as excessive precipitation and also steep foundation slopes may, in fact, also have been affected by the presence of zones of weaker, fine grained material.

## 5.5 Effects of Rate of Dumping

Data on the rate of consignment of waste material to a given dump has been identified as of importance by two major operations. The parameter of volume per day per metre of utilized length of crest has been selected as a measure of rate of consignment to a dump.

SITE A DUMPS

Rates of dumping have been plotted against time for Site A during 1986 for each dump shown on Figures 5.1 and 5.2. The rate of dumping parameter takes into account the length but not the position of the active part of the dump. An indicated steady rate of dumping may in fact correspond to a situation in which portions of a dump crest are being actively utilized, subsequently are experiencing high rates of crest deformation necessitating shut-down. Dumping operations are transferred to an adjacent segment of crest with a similar length.

The available records do not indicate whether or not the active segment of crest was shifted in this manner or, whether it remained in the same position. Monthly average dumping rates have been plotted for 1986 for site A dumps or Figures 5.1 and 5.2. Daily rates of dumping of up to 400 bcm's/m/day were achieved in the summer months.

In May and June, the length of crest utilized at two dumps which failed in June at Site A decreased significantly resulting in corresponding significant increases in the concentration of dumping. It is significant that during the remainder of the year; i.e. subsequent to failure, the total amounts dumped and the concentration of dumping remained at levels equal or higher than immediately prior to failure without resulting in further failure events.

Although it is possible that the apparent improved tolerance to high rates of dumping following the failure event is the result of a change in geometry and the removal of weak materials in the foundation, the significantly increased frequency of failure events and high rate events as shown on Figure 5.38 during spring months is additional evidence that springtime factors are a major, if not the major, set of circumstances resulting in high rates of crest movement or failure.

## SITE C DUMPS

At site C, the length of crest available is considered to be less variable than at site A. Thus, the parameter of BCM per metre per day shown on Figures 5.7 and 5.8 for site C is considered to be a valid indication of both the total volume being dumped and the concentration. The solid line on Figure 5.7 indicates the average rate of dumping in the immediate vicinity of the monitored location shown. The dashed lines represent significant dumping activity in adjacent dumping areas. The plots of rate of crest displacement against time show about eight prominent peaks. Comparison of the rate of dumping plot with the crest displacement plots does not show any clear relationship between increased rates of dumping and increased rates of crest displacement. A potential exception to this conclusion is the first peak experienced at the end of January when rates of dumping were relatively high. The peaks experienced during May and June, when the rate of dumping was relatively low, are considered to be the result of spring thaw. This may also be the case for the peak experienced at the end of March, noting that the mean daily temperature during March, 1986 was close to 0°C.

The highest rate of crest displacement experienced in the group A dumps at Site C was in the second half of August, 1986. This occurrence corresponds to a reduced level of dumping activity. It appears that the

rate of dumping was able to increase, immediately following the period of peak crest displacement, without resulting in further increase in the rate of crest displacement.

As for Site A the conclusion is that springtime activity seems to have a major influence on the permissible rate of dumping. Unexplained peaks shown on Figure 5.7 for Group A are suspected as being the result of consignment of fine waste material, although unfortunately no records are available.

Peak rates of crest displacement for Site C Group B dumps were experienced in early June with no corresponding increased rate of dumping. By contrast, increased activity, and a failure, were experienced in August during a period of significantly increased rate of dumping.

It should be noted that the dumps shown on figure 5.8 were commenced only in February of 1986, with toe slopes in excess of 30 degrees. Thus, the relatively high rates of movement shown during March and April may be the result of steep foundation terrain.

It should also be noted that the failure event in late August preceded the major precipitation event shown by several days. Clearly, precipitation was not a causitive factor in this event.

## EFFECTS OF BACKGROUND RATES OF CREST DISPLACEMENT

Figure 5.33 shows background crest displacement rate measured over approximately a two-week period, plotted against the estimated rate of dumping per metre of available crest length. Each data point is qualified with the known toe slope and face height. There is some indication

that steeper toe slopes and higher face heights result in higher background displacement rates for a given rate of dumping. Otherwise, the data show that for a reasonably consistent range of toe slope and dump height, there is an approximately linear relationship between the background level of crest displacement and the rate of dumping. The relationship indicated on Figure 5.35 appears to be sensible. If further data points are obtained it is anticipated that a family of curves for different toe slopes or dump heights will become evident.

## CORRELATION OF MONTHLY RATES OF DISPLACEMENT AND DUMPING

Figure 5.34 illustrates another approach to developing a relationship between rate of dumping and crest activity or displacement rate. Monthly averages of crest displacement rate and rate of dumping in terms of bcms per day/m crest length utilized has been plotted for each month of 1986 for the two dumps which experienced major failures during 1986. Although this method of calculation smooths out higher short term rates of crest displacement, there appears to be a clear relationship which depends on the time of year between the rate of crest displacement and the rate of dumping.

## DAILY CORRELATION PRECEDING FAILURES

### Site A

As a more detailed approach, we have examined the relationship between rate of dumping per metre utilized and rate of crest displacement in the months of May and June preceding the failures experienced at site A during 1986. Figure 5.35 shows groups of numbered points divided into one week intervals. The running average of dumping rate taken over a four day period is plotted against the rate of crest displacement experienced at the end of that four day period.

High rates of dumping do not appear to have necessarily resulted in high rates of crest displacement. In both cases dumping was halted altogether when the crest displacement rate exceeded 1.2 metres per day.

The dump depicted on Figure 5.35(a) shows a relatively high concentration of points below a crest displacement rate of 0.7 metres per day and indicates generally stable conditions under a wide range of rates of dumping.

No clear reason has been established for the migration of points representing the week preceding dump failure to the right of the critical shut-down level of 1.2 m/day, except that the toe slope was steepened slightly. A major rain event during Week 7 does not appear to have had a significant effect on the dump shown on Figure 5.35(a), but may have been a factor for the other dump shown on figure 5.35(b) which did fail at the end of Week 7. Poor material was identified as being consigned to this dump. Weeks 3 and 4 shown on Figure 5.35 correspond to the second half of May, 1986 and the onset of spring thaw, as indicated by the consistent elevation of the mean temperature above freezing level. In the early weeks of June, depicted by points 5 and 6 on Figure 5.35(b) it appears that an increased level of rate of dumping was tolerated without crest displacement exceeding the caution level of 0.7 metres day. It appears that subsequently in Week 7 the combination of poor material and heavy precipitation were key factors leading to failure.

Figure 5.36 illustrates dumping activity for the Group B dump at Site A shown on Figure 5.2 with a toe slope of 18 degrees. Significantly higher rates of dumping in the order of 400 to 500 bcms per metre/day of utilized crest correspond to measured rates of crest displacement in the range of 0.3 to 0.8 m/day until about August 23rd.

After this time, the rate of crest displacement increased and exceeded 1.2 m/day, resulting in shut-down. It is apparent that the rate of crest displacement then decreased in response to cessation of dumping activity. Figure 5.36 is included to serve as an example that peak rates of dumping are not unique, and depend upon other contributing factors such as precipitation or material quality, or possibly toe slope.

Site C - Group B

Data on rate of dumping at Site C is available on a weekly total basis, and therefore plots of daily rate of dumping versus rate of crest displacements are not possible. However, periods of inactivity in the vicinity of a specific monitor have been recorded on a daily basis, enabling an equivalent daily rate of dumping to be inferred. Figure 5.37 shows a plot of average daily dumping activity versus crest displacement rate for the month of August, 1986. The example shown is for a dump which failed at the end of August, as shown on Figure 5.8.

The parameter of activity is expressed as the running average over two days of the number of active dumping hours per day. In this way, it is possible to track the general level of dumping activity and the corresponding response in crest displacement rate. The dump characterized in Figure 5.37 is founded on relatively steep terrain with a toe slope of approximately 30 degrees and a face height of about 215 m. The numbers shown on the plot correspond to the number of the day in the month and begin at Day 8. The level of activity fluctuated from day to day.

As shown on Figure 5.8, the rate of dumping expressed in BCM's per metre used per day was the highest of the year, amounting to approximately 100 bcm's/m/day. From the beginning of dumping activity on

August 7 the average crest displacement rate increased to a range of 0.6 to 1.0 m/day. At the same time the overall rate of dumping was maintained. After August 19th, the crest displacement rates appeared to have decreased up to August 26th. During this time, there were only short periods of dumping activity.

In the absence of well defined documentation, we suspect that the relatively low crest displacement rates of less than 0.6 metres per day between August 20 and 26 give a misleading indication of dump stability. Clearly, the visual signs of crest deformation were deterring operations personnel from re-opening the dump. Although not shown on Figure 5.37, the rates of crest displacement at an adjacent segment of the dump, approximately 150 m distant, were consistently in excess of 2 m/day during the same period.

The foregoing attempts to correlate rate of dumping with rate of crest displacement have illustrated that rate of crest displacement does respond to rate of dumping. However, other factors such as quality of material and climatic conditions or seasons appear to be more dominant. Assessment of the conditions leading up to the failure event depicted on Figure 5.37 indicate that:

(1) ample warning of impending failure was evident to operations personnel who were able to reduce or shut down dumping operations in the affected area several days prior to failure; and

(2) suspected inappropriate positioning of the crest pin resulted in a misleading downwards trend in crest displacement until August 26th.

In conclusion, we consider that plots of dumping rate versus rate of crest displacement can be used to characterize the response of a waste dump to various contributing conditions. However, limitations of data quality have not permitted refinement of this method of characteri-

zation. One of our recommendations will be to adopt routine recording of daily rates of dumping, location of monitoring equipment, that is, configuration and pin location, together with such factors as quality of material, crest length actively utilized. With consistently well documented monitoring data, it should be possible to develop trends in the plots of dumping rate versus rate of crest displacement which are sufficiently consistent under a reasonably uniform set of conditions. This information would be useful as a means of predicting waste dump performance and improving planning. When such correlations have been established, it should eventually be possible to plan the rate and concentration of dumping in a given set of circumstances with a realistic expectation of the manner in which the dump will respond.

## 5.6 Effects of Climate

### GENERAL TRENDS

As discussed in Section 4.3.6 and shown on Figure 4.8, climatic conditions at each of the mine sites appear to be generally similar. The seasonal trends discussed in Section 5.1 clearly indicated an increase in dump crest deformation during the spring months. Figure 5.38 shows the frequency of occurrence of both failures and periods of high rate of crest deformation plotted against month of year. Clearly, the frequency of failure events and periods of high crest displacement rates is greatest in the months of April, May and June. The lowest frequency occurs during the winter and late fall months. The case history plots of crest displacement shown on Figures 5.1 through 5.9 for various mine sites each show prominent seasonal rises in rate of deformation or occurrence of failure events during the spring months. The data show that the rise of the mean temperature consistently above 0°C, or the relatively rapid rise in river flows corresponds to the onset of increased rates of crest deformation.

SPRING THAW

Response to the onset of spring thaw conditions is variable and appears to depend on the occurrence of other adverse conditions such as poor material, steep toe slope, and high rate of dumping. For example, the response of Site A, Group B dumps during late April and May is significantly greater than for Group A dumps during the same period in the absence of clearly defined factors such as poor rock quality. The dumps at Site C all exhibit marked response to the onset of spring thaw as indicated by consistent elevation of mean temperature above 0°C within one to two weeks.

The seasonal trends shown on Figure 5.9 for an abandoned dump at Site H over a three year period also appear to show a more subdued, but nevertheless evident upward trends in the rate of displacement during May and June.

RAINFALL

As shown on Figure 5.38 the frequency of events during the months of July, August and September is intermediate between the highs of the spring months and the lows of the winter months. Inspection of the precipitation records shown on Figures 5.1 through 5.9 indicates that the frequency of intensity of precipitation events in excess of about 25 mm is greatest during the summer and early fall periods. The frequency of failure events and high rate events was plotted against the average precipitation in the preceding week for the months of June through October. It was found that the largest number of events corresponded to average precipitations of less than 9 mm, as shown on Figure 5.39. It is our interpretation that precipitation of less than about 25 mm is probably not a significant factor in the failure event. Inspection of Figures 5.1 through 5.9 also suggests that the correlation between

high rates of crest deformation and occurrence of precipitation exceeding 25 mm is much higher than for precipitation events less than 25 mm.

LOW TEMPERATURE

The effect of ground freezing has been referred to in preceding sections. A recent case at Site C is considered to be the result of a rapid decrease in temperature in November of 1985, as shown on Figure 4.8, and the absence at that time of a significant cover of snow. We are aware of only two events in which this mechanism is suspected of being a major contributing factor in the failure. However, this factor should be anticipated as a possibility for future design purposes.

SNOW

The effects of incorporating snow into waste dumps is not clearly understood or documented. There are several suspected instances where the incorporation of snow is thought to have been a factor in a failure event. Observers at one mine site consider that there is a correlation between the position of a crest in winter time and the subsequent location to which a failure scarp breaks back behind the crest during a subsequent failure event. We are not aware of any observations of snow in the exposed upper segments of past failure scarps.

It is common that waste dumps on the windward side of mountain ranges are maintained largely free of significant thicknesses of snow on the dump face. Conversely, dumps placed on the predominant leeward side can accumulate significant thicknesses of snow. However, we anticipate that in general the action of consistently placing granular material on relatively thin veneers of snow cover is likely to generally displace that snow cover by incorporating the snow into the waste rock matrix.

We do not believe, and have not seen evidence to the contrary, that significant planes of weakness in the form of snow are commonly incorporated into waste dump structures. Nevertheless, we recommend that mine operators continue the current practice of minimizing the volume of snow incorporated into dumps. Areas prone to accumulation of wind blown snow should be avoided.

## 5.7 Characteristics of Dump Deformation

The object of this section is to present available information on the geometric patterns of deformation observed over dump faces and dump platforms. Various investigators at several mines have measured the magnitude and inclination at various points on the face and the platform of several dumps. This information is to used to confirm proposed mechanisms of dump deformation, as will be discussed in Section 7.

Available documentation of geometric profiles both before and immediately following failures have been assembled, classified and summarized.

The point at which large deformations are classified as failures is not clear. The Ministry of Energy Mines Petroleum Resources has defined major failure events which must be reported as those in which the material in the toe region is displaced in excess of 30 m. This distance is an arbitrary figure. Displacement of a single metre as the result of strain along the dump/foundation surface of contact would probably be the result of strains well in excess of 'failure' strain level. In fact, such levels of deformation, as the result of shear, would probably correspond to the full mobilization of available shear strength within the rock fill mass. Examination of relative distributions of vector

magnitude and inclination at different points on the waste dump surface, provides an improved understanding of mechanisms of deformation.

VECTOR INCLINATION

Table 5.4 summarizes the measured angles of inclination of the total displacment vectors at various locations on dump faces and crests. The overall average inclination, based on the available data, is 53 degrees. This value is the average of the values shown on Table 5.4 and is an indication of the total vector of displacement. As shown on Table 5.4, the inclination of vectors varied from as flat as 38 degrees to as steep as 67 degrees near the crest.

Data available for measurements of dump faces near the toe indicate that the inclination flattens. The range for the available data was 23 to 35 degrees. This latter range for the toe slope appears to be between the face angle inclination and the inclination of the dump/foundation contact.

There is no clear trend for the vector inclination to steepen or flatten with increasing distance behind the crest of the dump face. We consider that in cases where dumps are actively being utilized, or dumping has recently ceased, that the vector inclination will generally be in the range of 50 to 60 degrees.

VECTOR MAGNITUDE

Data are also presented in Table 5.4 on the variation of vector magnitude with position on the dump surface. There is no doubt that the magnitude of the total displacement vector decreases with increasing distance set back from the crest. Figure 5.41 shows the results of detailed measurements of dump surface deformation measured at an active

waste dump at Site C. We consider that the profile of crest deformation shown on Figure 5.41 is quite typical of deformations commonly observed at active waste dumps on moderately steep terrain.

The data in Table 5.4 indicate that toe slope is not a strong factor in determining the inclination of the total displacement vector at the crest. Vectors of crest displacement are shown on Figure 5.43 and are inclined at angles in the range 49 to 58 degrees. In this particular case, the angle of inclination appears to flatten with increasing distance behind the dump crest.

CREST SCARPS

The profile shown on Figure 5.41 also illustrates the formation of steep scarps between stepped segments of the dump platform. These steep scarps are inclined at angles is excess of 80 degrees to the horizontal over a height of 6 to 7 m. Similar steep scarps with heights of up to as much as 20 m, inclined at angles of between 60 and 70 degrees, have been observed. Steep, high scarp features appear to be more common during winter months, and are associated with the presence of a high proportion of relatively fine grained (silt and sand sized) particles.

Although the scarps between stepped segments of crest are steep, that is in excess of 60 degrees, the general inclination of the total displacement vectors are significantly flatter. This apparent difference in the direction of deformation would suggest that open cracks should form between adjacent blocks of a stepped crest profile. In the field, we do in fact see evidence of cracks widening, although this is often obscured by ravelling and localized collapse into the cracks. The magnified profile shown on Figure 5.41 also shows evidence of grabben-like settlement occurring. As discussed elsewhere, the steep scarps are

considered to be due to apparent cohesion, and to reflect a pattern of deformation which is localized in the upper portion of the dump. We consider that the measured vectors of crest displacement, generally inclined at angles of between 50 and 60 degrees, are more representative of the pattern of deformation within the main body of the dump.

EXTENT OF CRACKING

Figure 5.42 was obtained from measurements of the maximum extent of cracking behind the crest of waste dumps at sites C and E. The width of zone of cracking indicates the maximum observed extent of fissures behind the crest and does not necessarily imply that a large amount of differential settlement has occurred. Typically, the magnitude of settlement can be expected to decrease with distance behind the crest.

The variability of plotted data indicates that the extent of cracking is not only a function of waste dump height, but also of other factors such as material composition and probably toe slope.

Figures 5.43 and 5.44 show the decrease of crest displacement rate with increasing distance set back from the dump crest. Figure 5.43 shows the measured rate of vertical displacement in metres per day, measured by precise surveying techniques at Site B in November and December of 1982 and at Site E in December of 1986. The measurements for a particular dump were obtained almost simultaneously. The estimated dump height and toe slope is also shown for each dump. The data indicate that rate of crest settlement at a distance of about 10 m set back from the crest is approximately 25 to 50 per cent of the rate of vertical displacement close to the crest. The limited amount of data available does not show any clear trend which relates dump height or toe slope to the magnitude of vertical displacement or the way in which it decreases with increasing setback.

Figure 5.44 shows the variation of total vector rate of displacement with distances set back from the crest. In all cases, the rate of displacement decreases significantly with increasing set-back distance. The data for Site G were obtained for a dump which had a height of approximately 60 m and was founded on a toe slope inclined at approximately 20°. The data indicate that negligible movement was experienced at distances greater than 20 m back from the crest. It is significant that the toe of this dump was measured to have displaced approximately 25 per cent of the dump height, i.e., approximately 15 m beyond the original toe position. When plotted on Figure 5.42, the data point is significantly above the trend shown.

The data plotted on Figure 5.44 for Site B, with an estimated height of approximately 200 m, indicate, by extrapolation, that the approximate limit of cracking would be between 35 m and 40 m set back from the crest. When this estimated crack distance is plotted on Figure 5.44, the estimated range is significantly below the trend line shown. It is noted that the quality of waste rock at Site B is generally good.

In a similar way, the data points for Site G correspond to dumps with heights in the range of 100 m to 200 m approximately. From inspection of Figure 5.44, it appears that the extent of cracking is limited to between about 15 m and 25 m. The estimated range of crack and dump face height for the Site D dumps is also shown on Figure 5.42 and generally lies within the zone below the trend line shown.

In conclusion, the available data confirms that deformation of the platform behind the dump crest is greatest at points which are closest to the crest. Most of the available data confirms that the trend line, established by Golder Associates in 1979 as shown on Figure 5.42, is a reasonable upper bound in most cases which defines the maximum extent of deformation behind the crest of a waste dump.

FACE DEFORMATION

Figures 5.45 and 5.46 show the variation in magnitude and inclination in the cumulative displacement vectors measured at four locations over the upper 60-m length of face for a dump at Site G. This is the same dump for which displacement data behind the crest has been discussed in the preceding section. The magnitude and inclination are plotted against increasing distance measured down the face from the crest point. The data indicate that the inclination of the displacement vectors gradually flattens from a peak value in the range 50° to 60° at the crest, to the range of 20° to 40° at locations over the lower half of the dump face. The available data indicate that these trends were consistent over a six-day period. The data were obtained by surveying prisms set along two lines on the face of a dump which had recently been completed. The total amount of displacement appears to decrease with increasing distance down the face of the dump. The magnitude of total displacement was found to be approximately the same along both lines of prisms for a given distance down the face.

Although the data shown on Figures 5.45 and 5.46 were obtained from only one site, the indicated trends are compatible with (1) a concept of an upper active zone and a lower resisting zone and (2) observations carried out in the model tank in the laboratory (Appendix C).

We conclude that the available data demonstrate that the rate of change of deformation with distance is greatest in the vicinity of the crest of the dump, corresponding to the highest shear strains. At lower elevations on the dump face, the cumulative compression of underlying material, which must be some function of the thickness of loose waste material underlying, decreases. With increasing elevation above the

toe, the cumulative amount of compression over the height of the dump would increase.

## 5.8 Failure Events

This section summarizes the geometric characteristics of failure events for which information is believed to be reliable. Table 5.5 summarizes the geometric characteristics of observed failure mechanisms. In all of the cases studied, the upper part of the failure scarp is inclined at angles generally in excess of 50° to 60° over heights which may be as great as 30 m to 50 m. It should be noted that in most of the cases documented, there is no indication as to whether the final exposed scarp is coincident with the initial failure mechanism, or is the final event in a sequence of slip mechanisms which regress in the upslope direction. Figure 5.47 illustrates the general range of different mechanisms identified under different conditions.

As shown on Figure 5.47, failure surfaces for dumps on steep Type A profiles appear to pass along most of the dump foundation contact, steepening near the crest. A high proportion of the total dump volume is included in the failure mass and may move completely outside the limits of the original dump foundation area.

Failures of dumps on Type A profiles with flatter toe slopes, that is, toe slopes less than about 25°, involve larger volumes overall, and are the result of significant loss of shear strength close to or along the dump foundation contact in the toe region. The degree of loss of shear strength and the manner in which applied shear stresses decrease as toe displacement and slope flattening occurs, determines the extent of displacement of the slide mass. The extent of lateral displacement of material in the toe region affects the degree of loss of support of material in the upper part of the dump.

A limited number of visual observations of the failure mechanisms supports the concept of regressive slumping behind the crest of the dump. In one case listed on Table 5.5, the back scarp of the initial failure surface intersected the dump face below the crest.

In many cases, the upper part of the final failure scarp is depicted as being curved. The radius of curvature is described as being relatively large, although there is no compelling evidence that the surface is indeed circular. Further, some observers infer that lower segments of the failure mechanisms involved are also curved or circular. We have not observed any evidence which supports the speculation that the lower segments of the failure surface are circular or curved. In the case of Type A profiles on slopes of less than 25° which involve large volumes of material, and usually a regressive multiple failure mechanism, the lower part of the slide area is buried usually by a significant thickness of debris.

For Type B foundation profiles, the extent of regression by successive multiple slumps may be limited by the flattening of the foundation slope with increasing elevation. The same can be said for the complex profile, Type C.

Figure 5.47 shows two other conditions which have been identified. Although not documented, the condition of oversteepening of the upper part of the dump face, as shown on Figure 5.47(e), has been observed by the authors and reported by numerous mine operators. Slumping of the oversteepened upper segment of the dump generally occurs as a series of large slumps. Acceleration of the oversteepened mass down the dump face is not believed to occur. Where the foundation toe slope is relatively flat, we believe that the failure mechanism involves shearing quasi-parallel to the dump face. If the foundation slope in the toe region of the dump is relatively steep, then the slump mechanism may pass along

the dump foundation contact in the toe region. The extent of displacement of the debris beyond the original toe position is generally less than the dump height. Figure 5.47(f) shows a documented case of foundation spreading in which toe displacements did not appear to accelerate. The resulting final profile which was surveyed in some detail, shows a distinct flattening of the face at approximately the lower one-third point in the dump face together with extensive settlement of the dump platform behind the crest. The profile suggests that the double wedge mechanism was responsible for the movement, and that the toe wedge experienced downslope displacements of 10 to 15 m.

Table 5.6 summarizes the key elements of eyewitness observations of past dump failures. From the limited number of entries it is apparent that relatively few dump failure events have actually been observed. Comments on Dumps A8 and C2 suggest a failure process involving multiple mechanisms which regress up the slope from the toe region.

The unusually rapid loss of crest reported for Dump D4 is believed to be associated with the presence of both fine-grain material and seepage pressures. The comment on failure at Dump A2 that the toe "exploded" suggests that a substantial amount of elastic strain energy was stored in the toe region of the dump.

The early visual signs warning of impending failure observed preceding events at Dumps B1 and C7 confirm the common observation that visual signs, and difficulty maintaining a workable dump surface, almost invariably give operations personnel ample warning.

# 6.0 PERIPHERAL INFORMATION

## 6.1 General Behaviour

OBSERVED DEFORMATIONS

This section summarizes published observations of 'foreign' waste dumps in North America as well as overseas, at broadly similar waste rock disposal situations.

Harrison (ca 1975), studied the condition and behaviour of some 43 existing and abandoned waste dump sites in British Columbia. In particular, Harrison noted an increase over the period 1968 - 1974 in the occurrence of large crest displacements and crest failures. He identified that these crest failures differed significantly in comparison to other major failure events linked to such causes as weak foundations, or to the presence of fine grain material or snow in the waste rock.

Harrison carried out a detailed survey of crest and face deformations on an active dump at Mine Site C in 1972. A cross-section taken from this survey, showing vectors of total displacement, is shown on Figure 5.41. As discussed in Section 5.7, the vectors of crest displacement were inclined at angles of approximately 50° to 60° below horizontal. The pattern of crest deformation illustrated by Harrison in Figure 5.41 is typical of the observations by the authors over the past 15 years. In particular, the steep, nearly vertical, inclination of the step scarps is significantly different from the inclination of the total displacement vectors measured between the steep scarps. It is apparent that the shallower inclination of the movement vectors results in both the opening of tension cracks and the development of the graben-like features on the settling crest. Harrison notes that vertical settlement rates of up to 3 m/day were commonly experienced.

Gold (1986) describes the crest and dump face deformations experienced at a major dump at Site C in excess of 200 m high. The reported nature of crest deformations varies from one extreme of a single very high scarp set well back from the original crest location, to the other extreme of a series of up to 6 steps of varying height and width, set back from the original dump crest. The single major scarp could be up to 10 m high and set back up to 30 m from the original crest location. He confirms the author's observation that large bulges can form on the upper, middle, or lower regions of dump faces. These bulges may translate down the face of the dump, causing the type of large, wide crest deformations described earlier. Crest displacement rates, measured using two-stand type wire-line monitors, of up to 4 m to 5 m/day were commonly experienced without resulting in dramatic failure events. However, Gold notes that the toe displacements associated with these periods of high crest deformation are very large.

Blight (1969, 1981) described the behaviour of 50 m to 60 m high waste rock piles founded on a horizontal foundation consisting of a stiff fissured clay. The waste rock material, derived from gold mine operations, was a well-graded run-of-mine material having a maximum particle size of approximately 300 mm. Translational shear failure along a surface located parallel to, and at a shallow depth below, the dump/foundation surface of contact is reported as a result of relatively high rates of crest advance. Blight observed settlements of up to about 6 m within a zone extending up to 20 m back from the dump crest. The deformation of one of these dumps is illustrated in Figure 6.1.

The significant features of Blight's observations are the relatively steep displacement scarps, which were inclined at between 55° and 65°. Also, there is a clearly defined transition point in the deformed dump face at approximately the mid-height level. It should be noted

that foundation conditions similar to those described by Blight are not generally encountered at coal mine sites in British Columbia. That is, areas of flat terrain mantled by weak foundation materials which may be subject to significant loss of shear strength due to excess pore pressure generation under undrained conditions are not commonly selected for dump development.

Gonano (1980) and Richards, Coulthard, and Toh (1981) examined the behaviour of dragline spoil piles in Australia. The heights of these spoils were in the range of 60 m to 80 m, and the spoils were supported on relatively flatly dipping clay foundations. Gonano and Richards et al observed surficial deformational characteristics similar to those observed by Blight. The back shear surface appeared to be approximately planar, and curved slip surfaces were rarely evident. The spoil material was essentially granular in nature, containing sufficient fine grained soils in some cases to result in some apparent cohesion. Base translational shear failure close to the dump foundation surface of contact was due to the presence of excess pore water pressures. The primary back scarp was inclined at between 60° and 70° below horizontal. As found by Blight, the Australian dumps exhibited a prominent discontinuity in the slope of the deforming dump face. This discontinuity was generally located within the middle third of the dump face.

Campbell and Shaw (1981) report the use of a chart recorder to obtain continuous monitoring of a wire-line extensometer. The resulting record for an active dump indicated that dump crest deformations occur in a smooth or steady manner without clearly identified slip-stick episodes. In addition, the continuous recording showed negligible response to blasting activity at a nearby pit.

DUMP FAILURES

Nichols (1981) documented the behaviour of 12 current dump areas at a major coal mine in B.C. He found that waste dump failures occurred under the following conditions:

- Toe slopes in the range 25° to 37°.
- Continued dumping when crest movements were high (greater than 0.5 m per day)
- Where the toe slope increased with decreasing elevation from 14° to 30° (a type B geometric profile)
- Where foundation spreading or yielding occurred due to the presence of organic material.

The remainder of the dumps documented, which were stable, had heights in the range of 14 m to 160 m with toe slopes in the range of 0° to 20°.

Gold (1986) describes the changes in dump behaviour over a 3-year period, commencing with the initial stages when the dump toe was founded on slopes in excess of 30°, reducing progressively to slopes shallower than 25 degrees with advance of the dump face. In the early stages of dump development, Gold states that nine major failure events occurred which resulted in sturzstrom-type of runout. (Sturzstrom-type events are characterized by large runout distances which are indicative of dynamic friction angles significantly lower than the static friction angles expected under slow drained loading conditions.) As dumping activity has continued, shear deformations have resulted in the accumulation of large volumes of waste material on the slopes below the waste dump. The thickness of debris deposited on the slopes after approximately three years of operation was estimated to be between 20 m and

50 m. With a thickness of up to 50 m of material mantling the toe region of the main waste dump, it appears that rapid failures of the dump face do not occur, and that downward displacements in the region of the dump crest occur as gradual creep movements. Gold stresses that significant warning of impending failure events was always given either by visual observation on the crest or by wire-line monitoring prior to the dynamic movements that were experienced in the early stages of dump development, when the toe was supported on slopes in excess of 25°.

Typically, the failure zone would extend up to, or behind the crest, but not in all cases. If a significant amount of rehandle was present, the shear zone often intersected the dump face at a location below the crest. These observations support the general contention that failures initiate with large deformations in the toe region of the dump and propagate up the dump face. The presence of a significant amount of apparent cohesion due to the moisture held by capillary forces in the fine fraction of rehandled material has the effect of steepening the failure scarp so that its emergence on the face below the crest is quite credible. Gold also states that the greatest problems associated with low dump availability due to rapid displacements are the formation of large crest scarps which occur over a one- to three-week period in the springtime.

Both McCarter (1981) and Harrison (ca 1975) indicate that the level of noise emanating from within the dump mass can provide an indication of dump stability. Harrison reports that for a dump which subsequently failed, deep thuds every 30 - 60 seconds were clearly audible. McCarter also reported that large dump crest deformations preceding failure events were quite noticeable to truck drivers and other equipment

operators on the dump platform. Operations personnel are sensitive to increased difficulty in maintaining dump platform surfaces. McCarter also states that changes in vapour emissions (in cold weather conditions) may be a precursor to impending instabililty. This observation ties in with Golder Associates' observations of preferential snow melt in the middle one-third of dump faces during winter conditions. It is apparent that preferential convective draughting paths may be controlled by zones of differential shear and internal deformation. Alternatively, the presence of increased proportions of fine material within the upper one-third of the waste dump may be a more straightforward explanation. In this regard, Harrison reports excavating through the upper 13 m of an abandoned waste dump and finding significant evidence of compaction over that depth interval.

Harrison also refers to unsuccessful attempts to flatten steep failure scarps by either blasting within the dump or by pumping water into the dump. This comment confirms the general conclusions that blasting vibration usually has negligible effect on coarse granular waste rock piles. The absence of effect due to flushing with water may be due to the fact that concentrated inflow of water was drained locally through the coarser lower levels of the dump without generating pore water pressures, and without causing significant change in the moisture content of finer grained materials exhibiting apparent cohesion near the surface of the dump.

Based on the study of 43 sites, Harrison reached the tentative conclusion that few failures were experienced for dumps with a height of less than 60 m and that the frequency of failure was lower for flatter foundation slopes. Harrison also concluded that dumps supported on foundation surfaces with Type A profiles appeared to be more stable.

Dumps with sharply-curved crests in plan view tended to involve more volume when they failed than ones with gently curved or straight crests. This observation may reflect the relatively low sensitivity in the factor of safety to the point of emergence of the back scarp with respect to the dump crest. Where less lateral confinement is applied to the potential failure mechanism, relatively subtle changes in the statics of potential failure mechanisms may push the critical surface deeper into the dump.

## 6.2 Waste Rock Engineering Properties

A review has been made of published information regarding the engineering properties of rock fill materials similar to those contained within coal mine waste dumps in B.C.

Leps (1970) collated the results of relatively large diameter triaxial shear strength tests performed on a range of 15 different types of rock fill material. The data base used by Leps covered a wide range of material types in terms of particle strength, distribution of grain size, and degree of compaction or density. Leps sought to establish the relative importance of these factors upon the shear strength of the material. The triaxial tests were carried out under a wide range of confining pressures simulating overburden stresses resulting from up to 200 m thickness of waste rock. The results of this work are summarized on Figure 6.2.

Leps concluded that for a given rock fill material, with the same gradation, relative density and particulate strength, there is an approximately linear relationship between the available angle of internal friction and the logarithm of the applied confining pressure. Leps also concluded that rock fill materials composed of weaker particles, which were poorly graded, that is, uniformly or gap graded, and

with low density, exhibit lower angles of internal friction. Conversely, tightly-packed, well-graded rock fills whose particles are very strong, exhibit higher shear strength at a given normal pressure. In addition, more angular particles resulted in stronger rock fill materials. Leps also established that significant differences in shear strength were obtained by testing wet and dry samples. A rock fill composed of relatively weak particles was found to exhibit a dry frictional strength which was approximately 170% of the saturated strength. A relatively strong basalt exhibited a less marked difference, with the dry strength approximately 120% of the wet strength.

Leps also observed that failure strains at low confining pressures were between about 30% and 50% of those for high confining stresses. Poorly graded materials were found to fail at significantly higher strains than well graded materials. This implies that relatively large strains can be expected to occur where confining stresses are lower, or the material is of a more uniform in gradation.

Leps considered 'average' rockfill particles to exhibit unconfined compressive strengths in the range of approximately 20 to 70 KPa. This range would extend over approximately the lower half of the range of unconfined compressive strengths determined for rock particles in waste dumps at coal mines in B.C.

As an example, test data for El Infiernillo Diorite indicates a friction angle approximately of 46 degrees at low confining stress (equivalent to about 10 m of overburden) reducing to about 35 degrees under high confining stress (equivalent to over 200 m of overburden stress). The maximum particle size of the test specimen was between 100 and 200 mm with a void ratio of between 0.5 and 0.6. The gradation of

this material is shown on Figure 6.3 and is considered to fall approximately in the middle of the range of gradations extant at typical mine waste dumps in B.C.

Marsal (1973) carried out extensive strength and compression testing of a broad range of rockfill materials in a large triaxial apparatus capable of handling maximum particle sizes of about 200 mm. Marsal examined the inter-relationship between breakage of rockfill particles under shear and compressive stress and the shear strength exhibited by that material. Factors affecting this relationship including particle shape, size, gradation, relative density, and particular strength were documented and examined. As might be expected, Marsal found that particle breakage was higher for looser materials when tested in compression.

We have selected a number of the materials tested by Marsal as being potentially similar to the waste rock materials in waste dumps in B.C. The gradations for these materials are shown on Figure 6.3. Table 6.1 summarizes the key properties of the materials tested. The selected materials cover a range of gradations and degrees of packing ranging from loose to dense and from broadly to narrowly graded. In general, the maximum particle size of the test specimens was between 150 and 200 mm. Figure 6.4 shows the relationship plotted for these materials between angle of internal friction and the logarithm of the applied confining stress. Also shown on Figure 6.4 are the coefficients of uniformity and void ratios. It can be seen that more broadly graded materials with higher coefficients of uniformity exhibit generally higher angles of internal friction. Also, for a given coefficient of uniformity, materials with a lower void ratio, that is more dense, exhibit a higher angle of internal friction.

Marachi, Chan, and Seed (1972) carried out triaxial compression and shear strength tests on specimens with diameters ranging between 75 mm and 900 mm. Tests were carried out on a relatively weak and poorly graded argillaceous material for the Pyramid dam, and for stronger and well graded Amphibolite for the Oroville dam. Characteristics of these two materials are shown on Table 6.1 and their gradations are shown on Figure 6.3. The results show that compression increased with consolidation pressure. Also, the results indicated a modest increase in compression with increasing maximum particle size for the weaker argillite. This trend was less clear and pronounced for the stronger material from the Oroville dam. The results of triaxial shear strength tests showed decreasing angle of internal friction with increasing confining stress, similar to the results presented by Marsal and Leps. The results of the compression and shear strength tests for the Pyramid and Oroville dam materials are shown on Figure 6.5.

Marachi et al summarized the implications of extrapolating these data to materials containing larger particles sizes. If the particles are perfectly elastic and have equal moduli of elasticity and a Poissons ratio of 0.3, then the contact stresses between the particles are independent of the average particle size. Also, frictional resistance developed at the contacts is independent of particle size for these conditions. However, the foregoing assumptions are not valid for many rock types, particularly the anisotropic sedimentary materials which commonly contain planes of relative weakness. The effects of strength anisotropy are likely to be more significant for larger particle sizes than from smaller particles. Significant plastic deformation is always likely to occur with particle assemblages that have non-uniform moduli, are anisotropic, and have non-uniform particle size and shape.

More recently, Hribar et al (1986) carried out large scale direct shear tests on materials with maximum particle sizes of up to 300 mm, using a testing apparatus with load platten dimensions of 1.5 m by 1.5 m. The waste rock specimens consisted of durable sandstone and less durable shale materials obtained from coal mine operations in the eastern United States. The results for 'durable' sandstone with an unconfined compressive strength of 35 MPa and 'non-durable' shale with unconfined compressive strength in the range of 20 to 60 MPa are shown on Figure 6.6. The results are compatible with the general findings of other work. When plotted on Figure 6.4, the data points for both 'durable' sandstone and 'non-durable' shale are close to the line proposed for average rockfill by Leps. Figure 6.6 illustrates the significant difference between the results of large direct shear testing and smaller direct shear tests performed using an apparatus with a platten area of 0.15 m by 0.15 m. The difference in angle of internal friction measured between the the large and small tests ranges from approximately 7 to 8 degrees at low confining stress to between 2 and 4 degrees at high confining stress. The results obtained for the 'durable' sandstone and the 'non-durable' shale are similar. However, the degree of reduction in friction with increasing confining pressure is significantly less for the durable sandstone which has a lower void ratio. Also, the coefficient of uniformity is much higher for the 'sandstone'. When plotted on Figure 6.4, the results of Hribar et al plot close to the average established by Leps. From inspection of the tabulated values on Table 6.1 and the points shown on Figure 6.4, it is clear that gradation, characterized by coefficient of uniformity, and density characterized by void ratio, are major controlling factors in determining the effective angle of internal friction of the material at a given stress level.

Barton and Kjaernsli (1981) studied available results from triaxial compression and direct shear strength tests in order to establish the relative importance of particle shape, gradation, particle strength, overall particle size and confining stess. They established basic similarities between the behaviour of jointed rock masses and of rock fill. Based on the results of extensive research into the behaviour of jointed rock masses, Barton and Kjaernsli proposed an empirical relationship which relates the apparent angle of internal friction of a given rock fill to practical parameters which can be estimated.

The relationship is as follows:

$$\phi = R \log_{10}\left(\frac{S}{\sigma_n}\right) + \phi \text{ basic}$$

where

$\phi$ basic = basic angle of internal friction established from shear strength tests between polished surfaces.

R = an equivalent roughness factor which is based on the shape of the particles and the porosity, or degree of compaction, of the rock fill matrix.

S = the equivalent strength of the rock fill material. The equivalent strength, S, of the rock fill matrix is analogous to the joint wall compressive strength parameter developed for jointed rock masses. The equivalent strength is considered to be a function of the unconfined compressive strength of the parent rock material. The presence of micro-fractures in large rock samples

led Barton and Kjaernsli to introduce a reduction factor taking into account the average particle size.

$\sigma_n$ = the effective normal stress acting on the shear surface.

The relationships shown on Figure 6.7 were developed by Barton and Kjaernsli through the analysis of available strength test data. The equivalent strength and roughness parameters, S and R, can be determined by estimating the porosity and the average particle size of the rock fill matrix. Additional information required is an estimate of the unconfined compressive strength of the parent rock and the degree of the angularity of the particles. Barton and Kjaernsli were able to demonstrate a reasonable degree of accuracy in predicting the effective friction angle of materials from the Pyramid and Oroville dams. The method of prediction appears to be valid over a wide range of effective confining pressures. Also, they were able to reproduce the effect of varying the overall particle size of the rock fill matrix.

We consider that the relationship developed by Barton and Kjaernsli is the best available method of examining the sensitivity of frictional strength of waste rock to variations in composition.

Barton and Kjaernsli acknowledge that their empirical approach does not take into account directly the shape of the particle size distribution curve for the rock fill material. However, this may be compensated for, in part, by appropriate selection of values of porosity. As an alternative, Barton and Kjaernsli describe a full scale tilt test which enables field determination of the equivalent roughness, R.

Using the parameters documented by Hribar et al (1986) for 'durable' sandstone and 'non-durable' shale, we have found reasonable agreement with predictions using Barton and Kjaernsli's method. As found for the Oroville dam material, the predicted value of friction angle for the 'durable' sandstone was overestimated by 2 to 3 degrees at low confining stresses. At higher confining stresses, there was good agreement.

## Compression

The compression of rock fill materials is also dependent upon the applied stresses, the degree of uniformity of material gradation, the average particle size, the degree of compaction, and the strength of the parent rock material.

Parkin (1977) plotted the results of compression tests on a wide variety of material types and established approximately linear relationships between the logarithm of applied pressure and the logarithm of compression. It is noted that the materials tested were all crushed and the maximum particle size in all cases was less than 64 mm. Parkin concluded that dense, broadly graded, compacted rock fill would compress significantly less than loose uniformly graded dumped rock fill. Also, compression would reduce with increasing hardness of the particles and with increasing angularity. The upper bounds of the results obtained by Parkin indicate that up to 20 per cent compression could occur under an applied stress equivalent to 170 m of overburden. Parkin also looked at the crest settlement of some rock fill dams. He confirmed that higher crest settlements occurred for loosely dumped rock fill. However, it is significant that the total crest settlement for the worst recorded case at the El Infiernillo dam amounted to no more than 2 per cent of the overall dam height over a period of some 40 years. A significant portion

of the total compression of the rock fill may have taken place during the interval that the embankment was under construction.

Kenney and Yam (1978) cite the case of the Cogswell dam with a height of 85 m which settled a total of 6.2 per cent of its height, equal to 5.3 m. Approximately 30 per cent of the total settlement occurred as the result of a single extremely heavy rainfall of approximately 370 mm. The rock particles were a relatively low strength granitic gneiss. The rock fill was end dumped. Approximately 1.8 m of settlement occurred overnight following the rain event. Subsequent sluicing of the rock fill for a period of two months resulted in additional settlement of 1.2 m.

Clements (1984) summarized crest settlements of 18 dumped rock fill dams over intervals ranging between 2 and 40 years. Settlements of up to 2.5 per cent total dam height were observed. Kenney and Yam quote compression of up to 6 per cent in the base of the Venemo dam. Apparently, the fill was constructed in relatively thin lifts, consisted of moderately good quality granitic gneiss, and was subjected to sluicing.

The settlement of up to 17 m thickness of waste rock from a coal mine in the United Kingdom was documented by Charles, Naismith and Burford (1977). The material was generated as a result of a mix of truck/shovel and dragline operations. Holes were drilled through the spoil and encountered a mixture of sandstone and mudstone fragments with few boulders. A few 0.5 m size cavities were encountered, and the material was judged to be predominantly cohesionless. Wide variations in in-situ density and moisture content were determined. The void ratios varied between 0.25 and 0.7 with a saturation ratio varying between 20 and 100 per cent. Settlements of only 0.15 per cent were

measured for a 40 m thickness of this material over a 1 year period. Half of this settlement occurred in the upper 10 m thickness, probably due to the effect of precipitation. Subsequent inundation of the spoil due to flooding of an adjacent pit resulted in much larger settlements.

In conclusion, it is clear that pure compression even of very loose rock fill composed of weak particles is unlikely to amount to more than 10 per cent of the fill thickness. This upper bound is likely to be experienced under very adverse conditions of heavy sluicing. Given our knowledge of the composition of waste rock at mine sites in B.C., we consider that settlement due to compression is unlikely to exceed 1 to 2 per cent. For a vertical thickness of, say 100 metres, the expected settlement would amount to no more than 1 to 2 metres. This settlement could be expected to occur over a period of several months or years.

Material Degradation

Kenney and Yam (1978) summarize the influence of water on the compressibility of rock fill materials. The primary cause of increased compressibility due to wetting is the reduction in compressive strength of individual particles. The loss of frictional resistance at contact points is considered to have significantly less influence. They cite that the compressive strength of air dry rock is usually reduced by between 10 and 50 per cent as a result of submergence. They indicate an average value of 25 per cent. They state that larger strength reductions are to be expected for sedimentary rocks. The main effect of increasing moisture content is the propogation of tension cracks along pre-existing plains of weakness in the vicinity of point contacts where stresses are relatively high.

Research into the assessment of durability of waste rock materials is currently being carried out by the US Office of Surface Mining. Welch, Vallejo and Robinson (1986) discuss the relative merits of the slake durability test, and other methods including accelerated weathering and the sulphate soundness tests. In its present form, the slake durability test is not a sensitive indicator of potential for material degradation. We understand that consideration is being given to development of a modified slake durability test which will be more sensitive to actual material degradation under repeated wetting and drying conditions. Accelerated weathering tests in which material is subject to either 80 cycles of repeated wetting and drying, or to 35 cycles of repeated freeze and thaw correlates much better with field observations. As discussed elsewhere in this report, field observations themselves, if properly documented, appear to be a valid approach to the assessment of potential for material degradation.

## 6.3 Monitoring Methods

McCarter (1981) presented a comprehensive review of potential methods of monitoring mine waste dumps formed by end dumping and in excess of 60 m in height. He cites the practical difficulties associated with establishing durable instrumentation both at the crest and at the toe of active waste dumps. These problems include the large total magnitude of some movements that develop, harsh mountainous weather conditions, and heavy truck traffic. He states that rolling boulders from the surface of a 200 m high dump are sufficient to sever instrumentation cables buried 4 ft. below the surface at the dump toe. Large differential movements at the dump crest also virtually exclude any buried instrumentation such as extensometers. Poor visibility caused by dust, fog or snow also preduces the availability and reliability of

surveying methods. In addition, high noise levels due to equipment and wind tend to preclude the use of sonic monitoring methods.

As noted earlier, McCarter places considerable value on the ability of human senses to perceive changes. Inspections should be regular and sufficiently frequent to provide adequate warning of impending failure situations. However, continous surveillance is not a practical alternative. Observations by dump personnel including changes in the discharge of condensate during cold weather, or change in the colour of seepage exiting the toe of the dump represent inexpensive but significant data. McCarter has experienced several situations in which the sound of rocks fracturing in the toe region was clearly audible to the unaided ear. Apparently, the vibrations associated with rock breakage can sometimes be felt in the surface of the surrounding terrain.

Photogrammetric methods have been used successfully to monitor the movements of large dump areas to within accuracies of about 6 cm. This method may have some application to the long term monitoring of abandoned dump areas. Alternatively, simple methods such as the visual monitoring of lines of stakes may also be of practical value.

McCarter describes an extensometer consisting of a wire in a protective pipe buried under 2.5 m thickness of fill, and extending 30 m back from the crest to a measuring station. The cable was protected in a half inch steel pipe, and provided an indication of continued movement over a period of several weeks. Apparently, the protective pipe failed by corrosion. However, given the large differential shear displacements which are commonly observed at mine waste dumps in B.C., it is difficult to see the application of this type of extensometer.

We agree with the conclusion that the accuracy of individual measurements is less important than the ability to detect changing trends. Inclinometers have been successfully buried in dump foundations and are reported to have survived for up to three years. A marked increase in rate of rotation over a period of about four days prior to an episode of increased crest settlement has been detected. Acoustic geophones have also been installed in the toe inclinometers and also show a marked increase in events per minute over a period of several days prior to a settlement episode. Apparently the toe region of the dump is sufficiently free of mine traffic noise.

Any instrumentation, whether located in the toe region or near the crest, which involves connection to a remote sensing location, is highly vulnerable to large displacements. However, there may be some useful application of these methods for situations where dumps are founded on steep terrain, if shear through the foundation material is less likely to occur than through the face of the waste material.

## 6.4 Methods of Stability Analysis

Conventional methods of stability analysis using limiting equilibrium are well documented in the available literature. A variety of circular and non-circular potential slip surfaces can be considered. In nearly all cases, the potential slip mass is divided up into a number of vertical slices. Varying assumptions can be made in modelling the inter-slice reaction forces. Variable strength parameters can be assigned to different parts of the potential sliding surface. In nearly all methods, the factor of safety against potential instability is calculated as a ratio of the net resisting forces acting on the slide mass to the net destabilizing forces. In most cases, these conventional methods are appropriate provided that the user is confident that the

selected potential failure surface is appropriate, and that appropriate relationships to define the inter-slice forces have been selected. These methods of analysis do not directly take into account the patterns of stress and strain extant within a particular slope. The conventional methods assume that sliding shear resistance can be uniformly mobilized along the failure surface. In conventional civil engineering practice where relatively large factors of safety are selected in order to maintain stresses along the failure surface at levels well below the peak shear strength, this assumption is satisfactory and deviations from it are of little consequence.

One of the reasons for selecting relatively large factors of safety in conventional civil engineering situations is the need to provide an adequate margin over a long period of time against unforeseen combinations of adverse circumstances which either reduce the actual shear strength significantly below the assumed design value or which increase the applied shear stresses. By contrast, in the mining situation, many critical waste dump stability situations are of a short term nature. The adoption of relatively large factors of safety in civil engineering projects also has the desirable effect of maintaining low levels of strain and deformation. In short term mining situations such deformations are more tolerable. In addition, short term mining economics may lead to the acceptance of a higher level of risk of failure and a corresponding lower factor of safety than would be the case in conventional civil engineering. Exceptions to this would be situations where the consequence of dump failure is an unacceptable risk to mine infrastructure.

This section summarizes a number of methods of stability analyses which employ assumptions that are applicable to mine waste dumps.

Based on field observations of failure modes of rock dumps and also on simple small scale model tests, Blight (1981) developed a method of analysis assuming a wedge mechanism as illustrated on Figure 6.9. The development of active and passive resisting wedges, observed in his model studies is also illustrated in Figure 6.9. In his analysis, Blight assumed the the following:

- The backslope of the failure mechanism underlying the active portion of the wedge passes through the crest of the dump and is inclined at an angle of $45 + \emptyset/2$. This assumed inclination is a common assumption based on lateral earth pressure theory in which the major and minor principal stresses are oriented vertically and horizontally, respectively.

- The friction angle of the waste rock mobilized along the backslope was equal to the face angle of the dump.

- Blight's assumed inclination of the interface between the active and resisting wedges is not clear. Based on the inclination shown on Figure 6.9 for Blight's sand model, we presume that he assumed a relatively steep near-vertical interface. The results of Blight's analysis for a foundation slope range of 0 to 37 degrees are shown on Figure 6.9.

Brauns (1980) assumed that the orientation of the plane of maximum shear stress is parallel to the face of the dump in the toe region. The face of the dump is assumed to be at the angle of repose which is equal to the angle of internal friction of the waste rock material. Brauns uses Mohr's circle to determine the mobilized angle of friction on the foundation plane. A comparison of the results obtained by Brauns and Blight is shown on Figure 6.9. For low foundation slopes, the two

methods yield estimates of required shearing resistance along the foundation contact which differ by approximately 7 degrees. As the foundation slope approaches the face angle, assumed to be 37 degrees for the comparison shown on Figure 6.9, the difference diminishes to 0.

Campbell (1986) analyzed the stability of a two wedge mechanism based on his field observations, which refine the mechanism analyzed by Blight (1981). Campbell made the following assumptions:

- The angle of internal friction for waste rock is fully mobilized along planes AB and BD which bound the active driving wedge, as shown on Figure 6.8.

- The shear stress applied by the active wedge along plane BD acts in a downwards direction upon the resisting wedge BDC.

The positions of points A, B and D were varied to find the maximum value of mobilized angle of friction on the base plane, BC, of the toe wedge. The results of the optimization process indicated that the value of mobilized friction angle on the base of the toe wedge is not particularly sensitive to the position of point A, which tends to be close to the crest of the dump. This finding corraborates field observations of several shear planes commonly in the vicinity of the crest of the dump.

The optimization also showed that moving the position of point B significantly deeper into the dump than shown on Figure 6.8 resulted in lower calculated values of mobilized friction angles. The optimum inclination of the interface plane, BD was found to be relatively steep, in the range of 70 to 90 degrees.

Figure 6.9 compares the results of the analyses by Brauns and Blight with a typical result from Campbell's analysis for a foundation slope of 20 degrees. For this case, the results of Brauns' analysis are approximately 2 degrees higher and of Blight's analysis approximately 2 degrees lower. Differences between the results of Campbell's and Blight's analyses are due to differences in inclinations of boundaries of the active wedge. The differences between the results of Campbell and Blight diminish as the foundation slope increases between 20 and 30 degrees. At foundation slopes of less than 20 degrees, the mobilized base friction angle is less than approximately 29 degrees. If the available base friction angle is in the range of 33 to 35 degrees, which is typical for the surficial soils encountered at mine waste dumps in B.C., then the factor of safety against base sliding of the toe wedge is between 1.2 and 1.3.

More complex foundation profiles which necessitate the use of more than two sliding wedges or slices can be dealt with using a method developed by Hoek (1986). The method is based on the concepts developed by Sarma (1979) and calculates the horizontal acceleration required to bring the system to a state of limiting equilibrium. The method developed by Hoek has the advantage of permitting non-vertical boundaries between adjacent slices. This option is particularly amenable to the analysis of waste dump stability in which both observation and intuition indicate that the planes of maximum shear stress within the failing mass are inclined to the vertical. The results of Campbell's analyses have been successfully reproduced utilizing Hoek's method. The friction angle for all surfaces, that is slice sides and bases, within waste rock is set equal to the angle of internal friction for waste rock. Analyses, performed on a micro-computer, are iterated with varying values of friction on the base of slices in the resisting toe wedge, until a factor of safety of unity is obtained.

Vandre (1980) developed a chart for assessing the maximum rate of toe advance over saturated foundation soils of varying thickness. The method assumes that the rate of dissipation of excess pore pressure is proportional to the coefficient of consolidation, $C_v$, and inversely proportional to the square of the length of the drainage path. The practical application of this method to the stability of waste dumps advancing over weak, saturated, surficial soils is questionable since the coefficient of consolidation, $C_v$, the length of the drainage path, and the magnitude of excess pore pressures are very difficult to predict.

Caldwell, Moss and Vandre (1981) presented the Y = 0 method developed by Robertson (1976). Dividing an arbitrary slip mass into a number of vertical slices, the Y = 0 method calculates the net difference in the base shear resistance and the applied shear due to body forces, commencing with a single slice at the toe and progressively adding more slices up to the crest. In this way, zones within the sliding mass where inter-slice forces are tensile or compressive are identified. The factor of safety of each individual slice is calculated based on the body forces acting upon that slice including any net resistance which may be available from the slices below the one in question. Figure 6.10 shows an example of the analysis for a typical slope design. The positive values of the force P indicate that the slices between A and B are in compression and a net resisting force is available to support slices above. However, between points B and C, the P force is negative and the factor of safety against base sliding for individual vertical slices is slightly less than unity. This result is analagous to the full development of the active Rankine state in the upper part of the slope. In the example shown, a significant amount of cohesion results in the independent stability of slices above Point C. Thus, slices between Points B and C are close to a limiting state of equilibrium.

Since the net P force is positive when computation for all of the slices is complete, the net result is that slices between A and B provide sufficient resistance to permit the failed slices between B and C from displacing and forming a failure mechanism.

The Y = 0 method has been described because it supports the concept of an active zone in the upper part of the slope which is in a limiting state of equilibrium supported by a resisting toe wedge. The stability of the toe wedge depends upon the face slope and base slope of the toe wedge which affect the difference between driving and resisting forces. However, we recommend that Hoek's method using non-vertical slices be used to determine the distribution of shear stress and shear resistance along trial failure surfaces. In this way, inter-slice boundaries can be aligned to more closely simulate the planes where the ratio of applied shear stress to available shear strength is greatest.

Maksimovic (1979) analyzed the stability of a rockfill slope inclined at 1.5 horizontal to 1 vertical (33.7 deg), for three different materials whose angle of internal friction decreases with increasing normal stress. The characteristics of the materials are given on Table 6.1 and illustrated on Figure 6.11. Maksimovic used a stability analysis with vertical slices and tried different shapes of slip surface. He found that a circular surface resulted in a minimum value of factor of safety compared to the results for other non-circular trial failure mechanisms. Although vertical slices are used, and a uniform factor of safety is calculated for the entire trial failure mechanism, the results of increasing slope height for materials with non-linear failure envelopes are significant, as shown on Figure 6.11. With respect to waste dumps in British Columbia, we consider that the material from the Oroville dam most closely models waste dump material. The

results of Maksimovic's analyses would indicate that factor of safety decreases from 1.7 for a dump height of 10 m to approximately 1.4 for a height of slope of 1,000 m.

# 7.0 TECHNICAL ANALYSIS

## 7.1 Selection of Waste Rock Properties

### 7.1.1 Waste Rock Gradation

Figure 7.1 shows the results of grain size analyses carried out on samples of waste material obtained from the crest of several waste dumps. Overall, the results reflect the finer end of the spectrum of particle size distribution which can be expected near the crest of mine waste dumps. By contrast, the envelope shown for Site E obtained in 1982 was obtained for complete truck loads both by screening tests and from photographic analysis. Both are shown on Figure 7.1 as are the estimated relative proportions of sand, gravel and fines quoted by Nichols (1981) for re-handled waste material. Nichols reported that such material was observed to result in steepening of the crest of a 200 m high dump to approximately 40 degrees. Also, he reports that segregation is less clearly apparent, and that a period of reduced stability indicated by significantly increased rates of crest deformation, was associated with a period of prolonged dumping of re-handled material.

Campbell (1986a) presented a photographic documentation of the change of particle size with elevation on a dump face, as shown on Figure 7.2. A series of photographs substantiate the common observation that overall particle size tends to decrease gradually with increasing elevation above the toe. Campbell also reports on the results of scale model tests which demonstrated the following:

- In a typical column through the model, as shown on Figure 7.3 the overall gradation becomes progressively coarser with depth.
- Layers of coarser and finer material do occur within the body of the dump and are aligned parallel to the dump face.

- The overall gradations that reflect the average of coarser and finer layers within each sampling segment are such that downward migration of particles cannot occur.

Figure 7.4 shows the range of gradations as indicated from the results of particle size analysis for the finer end, as shown on Figure 7.1, and as estimated from visual observations of the coarse end of the spectrum. Also shown on Figure 7.4 are the grain size distributions for the materials used in large-scale direct shear tests by Hribar (1986). The spectrum of particle sizes of materials tested by Marsal (1973) and Marachi et al (1972) is also shown on Figure 7.4. These materials are considered to be the closest, in terms of particle size range and shape of gradation curve, for which large-scale shear strength test data are available.

The grain size distribution estimated by Blight (1981) for several failed gold mine waste rock piles is also broadly similar to waste rock materials at coal mines in B.C.

### 7.1.2 Void Ratio

Void ratio is defined as the ratio of volume of void space to volume of solid material. For a given particle size distribution, the void ratio provides an indication of the degree of compaction of the material. As discussed in Section 6, more compact materials with lower void ratios exhibit higher angles of peak internal friction than do loose materials with high void ratios. Also, lower void ratios tend to be associated with more broadly-graded materials where the smaller particle sizes fill in the interstitial space between larger particles.

Intact chunks of material were obtained from the crest of dumps at Site C in March 1987. Three of the samples obtained were frozen and

contained large amounts of free water, probably derived from trafficking of snow into the material matrix. The fourth sample, which did not appear to be frozen, exhibited a significant amount of apparent cohesion. Values of dry density and void ratio are given on Table 7.1. Calculated void ratios range between 0.12 and 1.32. The highest and lowest values may be the result of error due to the somewhat crude test method used in the field. However, we consider that the general range is indicative of conditions in the upper parts of the dump.

Direct measurement of density or void ratio of coarse segregated material in the toe region of waste dumps is not practicable. In 1981, Golder Associates carried out laboratory testing to determine maximum void ratio for a relatively uniformly-sized crushed gravel with particle sizes ranging between 20 mm and 40 mm. The results of three tests gave a range of 0.93 to 1.06 for the maximum obtainable void ratio. This range of values may be indicative of the maximum obtainable void ratio for coarser sizes with similar relatively uniform gradation, at low confining stress.

Analysis of the results of flow characteristics through an instrumented rock drain at Site C, Campbell (1986b), indicates that the void ratio for coarse waste rock beneath 50 m of waste rock is in the range 0.25 to 0.3. This assumes an average particle size of approximately 300 mm, with maximum particle sizes of up to about 1 m. Photograph 12 shows waste rock exposed by excavation at the base of a spoil at Fording Coal. The material shown has been overlain by approximately 50 m of material for a period of some 15 years. Visual estimation of the void ratio is in the range 0.3 to 0.4.

### 7.1.3 Shear Strength

As discussed in Section 6, the shear strength of rock fill is dependent upon the following factors:

- Particle shape.
- Strength of parent rock
- Grain size distribution
- Degree of compaction

The following sections relate the available data for mine waste materials in the above categories to the test data and methods of predicting shear strength presented in Section 6. The objective is to arrive at a reasonable practical range of material shear strengths applicable to waste dumps in B.C.

#### EFFECT OF PARTICLE SHAPE

Almost all mine waste materials, with the exception of a limited amount of overburden soil, consists of generally sharply-angular particles. Most of the available test data studied and presented in Section 6 relates to quarried rock fill for use in construction of dams. Referring to the predictive method developed by Barton and Kjaernsli, (1981) as shown on Figure 6.6, waste rock materials are considered as sharp, angular quarried rock.

#### EFFECT OF PARENT ROCK STRENGTH

The range of unconfined compressive strengths for mine waste rock material is shown on Table 4.3. The average values for different sites are variable. In addition, the relative proportions of sandstone, siltstone, mudstone and shale, is also quite widely variable, depending

upon the strata being mined. Further, there is probably a tendency for the hardest siltstone and sandstone particles to be more concentrated near the base of the dump.

The average value of unconfined compressive strength for the sandstone materials varies between about 110 MPa and 170 MPa. Average values for siltstone vary between about 60 MPa and 110 MPa, and the average for mudstones and shales was between about 30 MPa and 80 MPa. Table 4.3 indicates that the overall average unconfined compressive strength of material in a matrix of waste rock might be in the range of 50 MPa to 150 MPa.

## ESTIMATION OF WASTE ROCK SHEAR STRENGTH

The results of shear strength testing of rock fill samples similar to conditions within mine waste spoils were discussed in Section 6. The following key conclusions have been noted:

- Above a mean particle size of about 20 mm, the angle of internal friction is not greatly affected by particle size, provided that the shape of the gradation curve, the void ratio, and the particle strength are approximately the same.
- Uniformly graded materials give a lower friction.
- Materials with a higher density, and a lower void ratio, tend also to be better graded and to exhibit higher friction angle.
- Increased particle strength reduces particle crushing and increases friction angle.

It was demonstrated in Section 6 that a method of prediction of friction angle developed by Barton and Kjaernsli is reasonably accurate for materials similar to waste rock. Table 7.2 summarizes the range of rock fill matrices considered to be a reasonable approximation of actual

conditions within the mine waste dumps. The range of assumed particle strength is based on information on Table 4.3. The range of values assumed does not have a major effect on the predicted angle of internal friction. By contrast, the void ratio has the most significant effect. Possible values of void ratio are listed in Table 7.2. Normal stresses equivalent to overburden heights of 5 m, 50 m, and 200 m were assumed. The purpose of Table 7.2 is to examine the range of possible angles of internal friction at various locations and stages of dump development.

Field measurement of void ratio at various points within and on the surface of waste dumps is virtually impossible to perform except over the compacted trafficked dump surface. However, it can be argued that most of the material end-dumped on the face of a waste dump experiences some initial sliding and rolling, such that the void ratio is close to the critical value at which shear takes place without change in volume. For all points on the dump face with a relatively high void ratio of 0.6, the predicted angle of internal friction under a low normal stress is in the range of 41° to 42°. Dump faces have been observed as steep as 41° or 42° over the lower one-third of the dump face. The more commonly observed angle of repose of 37° to 38° may be the result of a dynamic component as materials dumped onto the face have a modest amount of kinetic energy. It is quite possible that were a dump face formed by end-dumping to 37° to be tipped gradually to a steeper angle, failure would not occur until an angle of 41° to 42° was obtained. It is possible that a void ratio as high as 1 might be obtained for uniformly-sized material near the toe of the dump. As shown on Table 7.2, the predicted angle of internal friction under low normal stress, would be 39°.

As the dump crest advances outwards, the confining stress acting upon material previously close to the surface of the dump face

increases. For a given void ratio, Table 7.2 shows the effect of increased normal stress in reducing the angle of internal friction. The process of compression due to self- weight within the mass of the dump may lead to additional shear deformation which results in reduction of the void ratio. Except at high stress levels, we conclude that the common assumption made in the past when analyzing waste dumps that the friction angle of the waste dump is always between 37° and 38°, is conservative. Although the face heights of some waste dumps are now as high as 300 m, the overburden thickness at points where shear stresses are high in the toe region of the dump may not exceed 50 m. As noted by Campbell (1986b), the void ratio beneath 50 m of waste rock at a rock drain composed of large-sized particle is inferred to be between 0.3 and 0.4. The results shown on Table 7.2 would indicate that the angle of internal friction for this material would be between 39° and 41°. With reduced overburden stress towards the toe, the void ratio might increase to a value as high as 1.0. Under low overburden stress, this would correspond to a friction angle of 39°.

Potential failure surfaces within the dump will pass through intermediate elevations where the particle size distribution of the rock fill matrix is well-graded and lies between the extremes found at the crest and the toe. The void ratio for these materials is expected to be lower, particularly at depths within the dump where shear and compression may serve to reduce the void ratio. The friction angle is not expected to be less than 38° and may be significantly greater than 40° where the void ratio and/or confining stress is significantly lower. Near the crest, confining stresses will be lower overall. The void ratio of freshly-dumped material may be relatively high, resulting in a friction angle of about 40°. The presence of more than about 10% to 20% of fine sand and silt sized particles in the rock matrix will tend to reduce this value to between 35° to 38°.

The presence of fines in the rock fill matrix, if partially saturated, will impart some cohesion due to capilliary suction.

## 7.2 Selection of Critical Failure Mechanisms

The selection of realistic trial failure mechanisms is the most important aspect of the assessment of waste dump stability. In the preceding section it was shown that the selection of an internal friction angle of 37° is conservative in all cases except for extremely high dumps. As discussed in Section 6.4, the method described by Campbell (1986), assumes a biplanar failure surface. The base slope is assumed to pass along the dump foundation contact. In most cases, this is an intuitively obvious location for the base shear plane where the foundation soils are identified as being significantly weaker than the waste rock material. The inclination and shape of the failure surface within the waste rock, whether planar or curved, is less easily argued. The inclination of the crest deformation vector, discussed in Section 5.7 is generally between 50° and 60° below horizontal. Blight (1981) observed the backslope angle in a model of a rock dump being inclined at 50°, as shown on Figure 6.9. The results of model studies carried out by Golder Associates, as part of this dump study, are included in Appendix E. The backslope was observed to be inclined at approximately 56° and to be planar. A second distinct shear plane was detected, defining the boundary between the resisting toe wedge and driving 'Active' wedge above. This plane was inclined at approximately 5° from the vertical.

The available model evidence indicates a planar shear surface which passes approximately through the crest of the dump. Field observations of slightly-curved back scarps, particularly near the crest, are considered to be due to a modest amount of apparent cohesion within the upper region of the dump.

The sensitivity analyses carried out by Campbell (1986) to determine the optimum position of the back scarp found that flatter backslopes passing well behind the dump crest resulted in higher factors of safety due to the increased sliding resistance available on the flatter backslope. Steeper back scarps resulted in less net thrust on the resisting toe wedge. It is noted that trial use of a slightly curved backslope with a large radius would yield similar results.

## 7.3 Stability Analyses

As discussed in Section 7.2, the available evidence of field observations and model tests indicates that trial failure surfaces within waste dumps can be modelled as a series of wedges with planar bases. In simple cases where the toe slope is uniform over a significant range of elevation, the double wedge analysis developed by Campbell (1986) is considered to be valid. For more complex foundation slope profiles, several wedges with planar bases can be analyzed using the multiple non-vertical slice method developed by Hoek (1986). The following sections describe the results of sample analyses using Hoek's method to demonstrate sensitivity of dump stability to various conditions:

- Variation of effective angle of internal friction on the backslope passing through the waste rock.
- Effect of a weak plane parallel to the dump face at some depth below the dump surface.
- Effect of raising a phreatic surface above the dump foundation contact.
- Effect of crest oversteepening or toe oversteepening.
- Effect of heavy truck loading near crest of waste dumps.

In the following discussion, a waste dump with a height of 200 m, supported on a toe slope inclined at 25°, with a face angle of 37°, has been assumed. Where the failure surface passes through the waste rock, it is assumed to be inclined at 50°. This is considered to be a reasonable angle based on observations and the results of Campbell's sensitivity analysis for a two-wedge system (1986). The failure mass was divided into 18 slices, as shown on Figure 7.5. With a conservatively assumed angle of internal friction for the waste rock of 38°, it was found that an angle of internal friction of 30° was required to be mobilized on the base plane. This result agrees closely with Campbell's method. The results of the analysis indicated that normal stresses on the base and side of slices increase linearly from both the toe and crest towards the boundary between the active and resisting wedges.

EFFECT OF VARYING WASTE ROCK SHEAR STRENGTH

On the basis of the possible variations in angle of internal friction for waste rock shown on Table 7.2, friction angles of between 40° and 45° were assumed for the backslope of the failure wedge. It was found that increasing the friction angle from 40° at the base linearly up to 44° near the crest had negligible effect. Increasing the friction angle to 45° over the whole of the backslope resulted in an increase in the factor of safety by a factor of 1.05.

EFFECT OF WEAK PLANE

In order to simulate the possibility of concentrated dumping of weak material on the dump face, the backslope of the the trial failure mechanism was flattened to 37°. It was found that for an assumed friction angle of 32° for the backslope, the friction angle required to

be mobilized on the base plane face to 32°. Thus, under drained conditions, the effect of dumping somewhat weaker material is a modest increase in the friction angle required on the base. However, as discussed in the following section, a small rise in phreatic surface above the base of the dump would result in instability.

EFFECT OF PHREATIC SURFACE

The required base friction angle was found to increase significantly with moderate elevation of the phreatic surface above the base of the toe wedge, as shown on Figure 7.5. An increase in the elevation of the phreatic surface above the base equal to 25% of the depth of overlying waste rock results in an increase in required base friction angle of 6°.

The effect of buildup of hydrostatic pressure beneath the crust of frozen foundation material is illustrated in Figure 7.6 For this example, it was found that the factor of safety against sliding along the base of the toe wedge was reduced from 1.3 under drained conditions, to approximately 1.0 under the hydrostatic pressure shown.

The above examples illustrate the importance of excess pore water pressure on dump stability.

EFFECTS OF OVERSTEEPENING

The trial wedge failure mechanism shown on Figure 7.5 was modified to simulate steepening of the upper one-third of the dump face to approximately 42°. Results indicate that the required base friction angle increases from 30° to 33° when the slope is steepened to 42°.

Analysis of the oversteepened wedge of material supported on a slope of 37° was also carried out. If the finer material in this wedge has a friction angle of 33°, an apparent cohesion of at least 10 kPa is required to maintain stability.

## BACK ANALYSIS OF STEEP CREST SCARPS

Steep scarps up to about 10 m in height, inclined in excess of 60° have been observed. Back analyses of this situation indicated that an apparent cohesion of approximately 6 kPa is required to maintain stability. The stiff consistency of some lumps of fine grain waste material compacted by vehicles on a dump crest indicates that apparent cohesion may be as high as 50 kPa.

Loss of apparent cohesion due to wetting or thaw will be greatest near the surface of the steep scarp and will result in a gradual recession of the crest and flattening of the slope.

## HEAVY TRUCKS NEAR CREST SCARPS

The effect of moving a fully-loaded 170-ton dump truck towards the crest of a 10 m high scarp inclined at 60° was evaluated. The results are shown on Figure 7.7. The results of this two-dimensional stability analysis indicate that with an apparent cohesion of 10 kPa, a factor of safety of 1.2 or greater is obtained if the rear wheels of the truck remain a minimum distance of 4 m from the crest. The same factor of safety is maintained if an apparent cohesion of 25 kPa was available with the rear wheels practically reaching the crest with a set-back of 1.5 m.

These calculations are presented as an example of sensitivity to truckloading. However, more detailed study of load-spreading characteristics is required before safe operating limits for large trucks in the vicinity of steep crests can be clearly defined. The effect of dynamic braking loads should also be considered.

The same truckloading was applied to the case of the steepening of the upper face to 42°. This loading made little difference to the factor of safety since most of the shear resistance is frictional.

## 8.0 DISCUSSION

This section discusses issues and trends which have emerged during the study.

### 8.1 Monitoring Data and Procedures

Monitoring information for a broad range of dump conditions has been analyzed in Section 5.0. The information includes both visual observations and measurements of deformation.

#### VISUAL METHODS

Visual monitoring is common to all operations in British Columbia, and it is probably fair to say that experienced operating personnel take visual signs very seriously. As a result, we are not aware of any major failures of active dumping platforms which have occurred without warning. Unfortunately most of the observations made by operating personnel on a day-to-day basis are not recorded. However, as operating tools, this does not detract from the value of these observations and qualitative assessments by experienced operating personnel.

During the course of this study we have attempted to draw together available observations of crest deformation which can be matched with numerical data such as recorded crest displacement provided by monitoring data. It is evident that people such as bulldozer operators and crest watchmen are able to sense changes in behaviour of the upper portion of the dump, i.e. they are aware of the occurrence and the comparative rates of development of the cracks, and scarps produced by

crest movements, and they are cognizant where it becomes perceptibly more difficult to maintain a level dumping platform.

It is clear that the face slopes of active waste dumps comprising predominantly fine-grained material founded on steep terrain undergo noticeable deformations in the form of bulges over portions of the face. Development of these bulges may result in steepening of segments of the dump face to 40° or slightly greater. These bulges have been observed to translate down the length of the waste dump face. Routine measurement and observation of this deformation has not been carried out to date. Reliable measurements of such changes in the face profile are difficult to obtain, mainly because of the difficulty in finding a suitable vantage point which provides a consistent view of the dump face in profile. The perspective provided by a vantage point at the crest of a waste dump generally does not provide a good picture of the relative deformations occurring over the dump face.

We are aware of only four failure events which have been observed. Many of the failure events have occurred during hours of darkness.

## MONITORING MEASUREMENTS

The total vector of crest displacement is generally inclined at an angle in excess of approximately 50°. In addition, it has been observed that the magnitude of crest displacement decreases significantly with distance set back from the crest. It has also been observed that the magnitude of displacement on the face of a waste dump may decrease slightly with increasing distance below the dump crest. Various types of monitor utilized at mine waste dumps in B.C. are summarized on Table 4.1. It is apparent that a general trigonometric

relationship between magnitudes of total vectors and their horizontal and vertical components can be calculated based on an assumed average inclination of the vector of crest displacement.

In our review of the data and our discussions with mine personnel, it is apparent that some failure situations are not immediately detected using the wire-line methods for the following reasons:

- With a single-stand type wire-line, the crest pin may be anchored above a failure scarp.
- Positioning of the monitor device in relation to the area of active dumping is also important. We suspect that in some situations, monitors inappropriately placed in the periphery of an active area may give the misleading impression of stability. However, this possibility can probably be countered by the argument that visual indications will generally provide sufficient warning. The main exception to this conclusion would be the combination of particularly fine-grained material and wet conditions.

It is common practice to increase the frequency of measurement, i.e. reduce the response time, as the rate of deformation increases beyond a pre-defined critical point. The configuration and positioning of wire-line extensometers has a significant effect on the magnitude and interpretation of the data. A wire-line extensometer has been used to monitor movement at the toe region of a waste dump on occasion. However, due to the continuing advance of the toe of the waste dump generally downslope, this kind of measurement is generally not practicable due to impact of the monitoring point by rolling boulders.

Use of hydraulic settlement gauges merits some discussion. A trial of this method was carried out at Line Creek in the fall of 1982. A comparison with wire-line extensometer and survey measurement indicated

that the method does provide a good indication of movement at the crest and is more accurate than the wire-line when crest settlements are high. However, the system in its form when tested appeared to be less robust and reliable under harsh exposed winter conditions than the wire-line extensometer. Also, the hydraulic method is somewhat more complex, requires more maintenance, and therefore is less readily accepted by operations personnel. A broken wire-line is much more readily repaired than a broken u-tube. Also, the practical operating range of a hydraulic monitor is limited to approximately two to three metres of vertical displacements. The wire-line extensometer is capable of tracking much larger downslope movements.

Conventional survey levelling has been used at two operations to detect changes in elevation at various points on a dump crest platform. Frequency of measurement is variable depending on the amount of activity at the dump, in turn based on visual observation. Frequent measurements, i.e. several times per shift, could be taken during periods of high activity, with the objective of predicting a potential failure event. However, in general, the cost of providing routine frequent survey measurement is not justified. The quality of information obtained is equal to or better than that obtained using the wire-line extensometer provided that survey pegs remain undisturbed. The method has the disadvantage that operating personnel are not involved and may not receive immediate feedback, i.e. response time is relatively long.

One mining operation has experimented with using a rotating laser beam to provide rapid levelling measurements at a number of points on a dump crest within a short period of time. This method could have the advantage that operating personnel can be involved and use the device.

However, the laser method is not practical in foggy conditions, may be subject to poor reliability during harsh winter operations, and is not practical for ramping dump platform crests.

Total vector surveys of points on both the dump crest and face slope have been carried out at several operations. This method permits tracking of changes in magnitude and direction of the displacement vectors at various points on the crest platform and on the dump face. Clearly, face monitoring is practical only on the faces of inactive dumps. The frequency of measurement has depended on the level of activity and the urgency with which measurements are required, but can be up to several times per shift if the dump is highly active. The method is expensive in manpower, and again does not involve operating personnel or give them rapid feedback necessarily. However, the information gained is highly valuable in providing information on the relative distribution of deformation and its direction.

Maintaining and protecting wireline monitors or survey pegs within constricted active dumping areas is difficult. However, we consider that in nearly all situations a practicable method of monitoring crest deformation which will yield meaningful data is available.

Automated tracking of survey stations may eventually become a practical, and economical option.

## DATA PROCESSING

Procedures for recording and processing monitoring data are variable. Routine extensometer readings at intervals of several hours are

normal. The readings are usually obtained by a consistent group of operations personnel who recognize their importance.

Significant errors in calculating the rate of crest displacement often result in a "spiky" plot of rate versus elapsed time. This error is usually due to inaccurate recording of time intervals between readings. Smoothing of the monitor data by taking a running average of several readings seems to result in more consistent and realistic trends. We consider that dumps do not deform in a slip-stick manner.

Two operations transfer manually read extensometer readings to a computer database for processing within a few weeks for documentation purposes. In general data is manually recorded on specially devised forms and changes in trend are communicated verbally. We consider that the verbal system of communications warning of potential hazard, supported by visual inspection of dump behaviour, is satisfactory.

As micro-computers become more accessible and user-friendly we anticipate that monitoring data will be processed as it is obtained. The results could be displayed as a current plot of either cumulative displacement or rate of displacement against elapsed time. However, we consider that these improvements will not lessen the importance of visual observations and appropriate positioning of crest monitoring devices.

## CAUTION CRITERIA

Over the past several years the operators of large dumps have experienced low availability due to crest displacements exceeding the longstanding and commonly accepted critical rate of 0.5 m per day.

By documenting rates of crest displacement preceding failure events individual operators have been able to justify increasing the critical rate to a higher value applicable to a specific dump. We support this rational aproach which has been discussed in detail in Section 5.

We conclude that critical rates above which dumpng must cease in the range of 1.0 - 1.5 m/day appear to ensure sufficient warning prior to failure for most well established dumps.

For all new dumps, particularly on steep slopes or involving weak material, we recommend a critical rate of 0.5 m/day. This value can be reviewed following a period of monitoring.

## 8.2 Modes of Failure

### WEDGE MECHANISM

Often, it is evident that the failure consists of multiple events initiating in the toe region and propagating as series of slivers or slabs. The extent of loss of dump crest is generally observed to be relatively small in relation to the height of the dump. In fact, the initial mass movement may not involve the crest, although the resulting steep scarp at the top of the initial failure event will tend to regress back towards and somewhat behind the original dump crest.

We have found some similarity in mechanism of failure between B.C. Coal mine spoils and those elsewhere. The main similarity is the development of apparently bi-planar wedge boundaries and acceptance of the concept of non-uniform factors of safety along segments of the failure surface. However, foundation slopes elsewhere are generally flat, or near flat, that is less than 10°. Materials are predominantly

much finer, and the generation/dissipation of excess pore water pressures, both in the foundation and the spoil material itself, are dominant factors in the failures. The main value of the comparison is to illustrate the similar bi-planar failure mechanisms.

The results of model tests also confirm the concept of a sliding wedge failure mechanism with planar boundaries. More complex multiple wedge mechanisms can be utilized to analyze complex foundation slope profiles.

Relatively rapid imposition of compressive and shearing stress due to dump construction or downslope translation of debris from a failure will result in the generation of excess pore pressures. The rate of dissipation of the excess pore pressures will depend upon the permeability of the material and the length of the drainage path. For example, a thin surficial veneer of fine grained material interspersed with layers of more pervious granular soil tend to be less saturated and any excess pore pressures resulting from contraction of the soil matrix tend to be rapidly dissipated.

By contrast, deep deposits of saturated fine grain soils in a lightly consolidated state may be highly susceptible to loss of effective shear strength due to excess pore water pressure buildup. This latter condition is not characteristic of foundation conditions at mine sites in British Columbia.

Failures due to the presence of weak foundation soils occur with a relatively low frequency.

Little data is available from monitoring of pore water pressures in dump foundations as this has not been common practice. Also, the development of a phreatic surface, which is elevated significantly above the dump foundation contact, has not been observed or recorded.

The presence of extensive areas of lightly consolidated, saturated, fine grained soils adjacent to the toe of a waste dump which fails can have a major effect on the subsequent behaviour of the unstable mass. Rapid loading imposed by the translating slide debris over a significant area has been observed to result in high mobility, that is, low apparent base frictional resistance. In several instances, this effect appears to have resulted in the movement of slide debris in a direction sharply oblique to the initial direction of failure, along the base of the valleys with slopes of less than 10°.

From the available case history data, we are aware of two other types of foundation-related failure. As noted earlier, development of hydrostatic pressures beneath the crusts of frozen surficial natural ground appears to have been a factor in two significant failure events. There is also some evidence to suggest that subtle reductions in apparent cohesion caused by an increase in moisture content may be sufficient to shift a quasi-stable dump into an unstable state.

Failures have also been observed to occur as the apparent result of oversteepening of the dump face in the crest region, that is, the upper one-third of the face. In this type of situation, observers at the crest are unable to see the toe of the dump, and identify a prominent bulge in the upper part of the face. This type of occurrence is usually, if not always, associated with a relatively high proportion of fine grained waste material being consigned to the dump at that time.

## 8.3 Factors Affecting Stability

### TOE SLOPE

The results of the study indicate that stability is reduced with increasing toe slope. In the absence of other adverse factors it appears that the incidence of dump failure or of high rates of crest displacement increases significantly when the toe slope exceeds about 25 degrees.

### DUMP HEIGHT

Much of the data considered in this study has been obtained for dumps with heights in the range 150 to 300 m. Clear relationships between dump height and behaviour have not become apparent. This poor correlation is considered to be due to the stronger influence of other factors.

In the absence of other adverse factors, crest deformation might be expected to increase with dump height due to increased bulk compression. It may be significant that limited available data for dumps with heights in the range 50 - 150 m indicates crest deformations about an order of magnitude lower than for dumps 150 m - 300 m high.

### CREST ORIENTATION

As shown on Figure 4.5, it can be seen that the direction of advance of the dump crest in relation to the orientation of the natural topographic contours affects the proportion of the supporting toe region of the dump which is founded on steep terrain. As noted in Section 5.2.4, the natural topographic slopes at the higher elevations at Site B

are in excess of 30 degrees. Two-dimensional stability analysis of sidehill fills placed on such slopes will generally indicate a factor of safety close to unity against base sliding. The evidence gathered in this study from case histories of past failures also indicates that the frequency of occurrence of major failures is highest for toe slopes exceeding 25 degrees. The value of controlling crest advance direction to ensure that the toe of the waste dump is always supported on slopes which are generally flatter than 25 degrees appears to have been demonstrated at Site B. As illustrated on Figure 4.5, advancing the crest of the initial platform once the toe has reached the lower flatter slopes in a direction parallel to the steeper topographic contours of the natural valley can be effective in maintaining a slope in the toe region of less than 20 degrees. By allowing the direction of crest advance to deviate to an angle which is oblique to the natural topographic contour, significant segments of the toe of the dump will be placed on steep topography, resulting in potential instability. We consider that the absence of major failures over a four-year period at Site B with repeated development of new platforms on steep natural topography demonstrates that appropriate control of crest advance direction is valuable in precluding dump failure as a result of inadequate support provided by the toe region of the dump on steep foundation slopes. By contrast, failure event number 62 appears to have occurred as a result of advancing the dump onto slopes in excess of 30 degrees in a direction normal to the natural topographic contours.

## FOUNDATION CONDITIONS

The same relatively narrow range of material property parameters appears to be applicable to most foundation areas. Spacial density of sampling from foundation areas during design investigations does not

usually permit narrowing of this range. The impracticality of higher density of sampling and testing can be compensated for by visual judgment and experience and selection of conservative parameters for design, which fall at the lower end of measured values. With these considerations in mind we have not generally drawn major distinctions between foundation soil types at different sites.

Detailed statistical treatment of foundation strength parameters for use in stability risk assessment calculations is not generally considered to be worthwhile.

We consider that selection of an appropriate range of possible foundation material parameters should depend on experienced judgement based on detailed site reconnaissance and carefully selected sampling locations.

The permissible factor of safety should reflect the sensitivity of dump stability to the expected range of foundation strength parameters.

PRECIPITATION

Collected data clearly indicate relationships between periods of precipitation in excess of about 25 mm and/or spring snowmelt with increases in rate of deformation.

EFFECT OF SNOW

Samples collected from the crest region of dumps in B.C. during the winter months indicate that considerable volumes of water can be incorporated into the fill.

Snow packed and mixed into the upper surface of the dump has been observed to result in a very high moisture content in samples of frozen crest material. In situations where dumping is essentially filling in continuously slumping crest areas, it would seem to be possible that frozen material with a high moisture content is incorporated in a significant zone of the upper part of the dump. Ice has also been observed in the toe region of several waste dumps. We are not aware of any reported situations where a significant layer of snow has remained intact as a potential plane weakness within the dump. It is general practice to avoid pushing cleared snow over the edge of waste dumps. Wind action in the mountainous regions also tends to reduce the thickness of accumulated snow in many of the dumping situations. In addition, we consider that the action of dumping material on a snow- covered slope tend to disrupt and mix any accumulations of snow within the rock matrix.

We anticipate that frozen material does not form a continuous mass but rather a series of zones interspersed by significantly dryer, more pervious material. In general, we believe that melting of zones with high moisture contents occurs in a discreet, somewhat erratic manner such that planes of weakness forming a continuous potential failure mechanism are not created.

It should be emphasized that linkages between weather conditions and changes in ground water flow patterns will be extremely complex, variable across individual dump sites, and very difficult to predict with any precision. Ground water seepage discharge appears to have played a major role in the behaviour of relatively few dumps. Fluctuating patterns of precipitation and temperature and snow pack dissipation

are expected to have subtle but significant effects on material parameters such as apparent cohesion due to capillary tension, or bulk density.

WASTE ROCK QUALITY

Discussions with mine personnel led to the conclusion that only very general retrospective correlations between lithology and quality or nature of waste rock being dumped at any one time is feasible. Records of the quality and nature of waste rock consigned to a dump at any particular time are either very general or intermittent. In going through past records we have attempted to pick up on any comments concerning waste material quality that are relevant to dump behaviour. However, the task of going back through mine status records at any given time, correlating that with known lithology from relative pit floor elevation on available stratographic sections, and coming up with estimates of quality of material at that time has not been considered practical by any of the mines.

This type of information appears to be an important factor in waste dump behaviour. It has been possible to draw a broader distinction between the general quality of waste material produced at different mining sites. For example, the quality of waste rock generated at Sites B and F is clearly much higher than the general case at sites D and G.

Development of an accurate statistical picture of the contents of a waste dump is not practicable. Future prospective studies in which suitably detailed monitoring of material quality and quantity and spacial distribution within waste dumps can be monitored, may permit a more accurate picture of the composition of a given dump to be deter-

mined. Subsequent behaviour due to the presence of weak layers may then be correlated.

Whether or not prospective prediction of waste rock quality is practicable in the future will depend upon the variability of the mining sequence and the mine lithology itself. Certain selected mining situations may be amenable to this.

In discussion with engineers from the U.S. Office of Surface Mining, it is clear that the practicality of applying the durable vs. nondurable criteria in a prospective way, based on submitted data from tests on core samples is of very questionable reliability.

DUMPING RATE

As with documentation of material quality, records of rate of disposal are limited. Where possible this type of data has been acquired. The reliability and frequency of measurement is quite variable but broad comparisons between rates of dumping at different operations have been made and linked to the degree of activity monitored at the crest of the dump.

The data and correlations presented in Section 5 indicate that the behaviour and stability of waste dumps is strongly linked to quality and rate of dumping. We consider that relatively inexpensive routine documentation of these parameters may allow individual operations to improve scheduling of dumping based on estimates of anticipated rock quality and other adverse factors likely to reduce dump stability.

## 8.4 Effect of Blasting

We have not encountered any documented instances of blasting having an adverse effect on the stability of waste dumps. The short duration and the generally coarse, pervious nature of waste rock does not result in development of a plane of weakness within the waste rock which is sustained for a significant period of time during which large strains might occur. The generally loose state, and thus relatively low density, of the waste material tends to result in rapid dissipation of blast energy. Blasting has been carried out at major dumps on the Fording Coal and Quintette Coal property in attempts to flatten oversteepened crest segments. Although the blasting was partially successful in flattening the oversteepened crests, we understand that significant changes in rates of crest deformation were not detected. Monitoring of blast vibration at one of the sites by Golder Associates in 1985 found that relatively rapid spatial dissipation of blast energy occurred. Loss of foundation support due to generation of excess pore water pressure in loose saturated fine grained soil as a result of blast vibration may be a possibility.

## 8.5 Long-Term Stability

This section discusses factors which should be considered in assessing the long-term stability of waste dumps.

### MATERIAL PROPERTIES

Documentation of long-term changes in material properties within the body of a waste dump is limited to two cases. The first case is illustrated in Photograph 12 in Appendix A. Although some fracturing of individual particles in the base of the dump had occurred, the excavated base layer appeared to be free of material which had disintegrated. We

understand that this dump comprises material which is broadly representative of typical coal mine waste rock. This spoil has been in place for over 15 years.

The second case is a rock drain, formed by end-dumping, which has been monitored over the past 5 years. Although a significant reduction in hydraulic conductivity was detected initially subsequent monitoring has not indicated further detectable change. The measured change in hydraulic conductivity appears to be a function of the thickness of overburden.

We consider that changes in the humidity and temperature of the atmosphere surrounding rock particles in the body of a dump occur very gradually. Such conditions do not result in rapid degradation even of susceptible materials such as mudstones. Buried snow and ice may take several years to melt.

In order that mass stability of waste dumps be significantly reduced one of two changes must occur:

either (a) the effective strength of the waste rock or foundation must reduce significantly;

or (b) poor drainage must cause a phreatic surface to rise significantly above the foundation surface.

Colluvial material found on the slopes of waste dump areas typically consists of a well graded assortment of particles of weathered rock. This material has been exposed to the elements for a relatively long period of time. These conditions are much more severe than at

depth in the body of a waste rock dump. Typical angles of internal friction for colluvial soils are in the range of 33 to 38 degrees. Thus, we conclude that long-term reduction in the strength of waste rock in a dump is likely to be relatively minor.

Assuming that major surface runoff is directed away from a dump the only cause for a rise in phreatic surface above the foundation contact would be seepage discharge. The phreatic surface would not rise if the coefficient of permeability of the waste material was significantly higher than for the foundation soil. In most situations we anticipate that this will be a valid assumption.

Thus, we conclude that long-term changes in material properties at depth within the mass of a dump are unlikely to radically reduce stability.

Waste dumps are usually located away from areas of major seepage discharge. We are unaware of any instances of abandoned waste dumps undergoing major dramatic failure.

LONG-TERM CONFIGURATION

In general, the toes of most abandoned waste dumps are supported on slopes which are relatively flat i.e., less than 10 degrees. Assuming conservative parameters of 33 degrees for the angle of internal friction for both foundation and waste rock the indicated factor of safety is at least 1.5. This assumes that the dump face is inclined at an overall angle of 27° degrees, as is currently required for abandonment.

In the event that material within the dump experiences modest loss of shear strength such changes are likely to occur gradually. Corresponding gradual strain within highly stressed zones could be expected. Dramatic rapid failure is not likely. Rather creep deformation typically observed in natural slopes is likely.

This downslope creep deformation is most likely to occur near the face of the dump where degradation of particles is more advanced. Saturation of this material conceivably could cause localized more rapid slumping. This slumping is unlikely to result in obstruction of overall base drainage.

Construction of wrap-around dumps would tend to limit the extent of shallow slumping. Also, revegetation of weathered surficial material will tend to reinforce the surficial soil cover and impede erosion or slumping.

In the future we anticipate the need to construct major dumps across valleys. We consider that obstruction of drainage courses by high dumps will require very careful consideration. Further research is required with emphasis on field instrumentation of major rock drains.

We consider that further assurance of the long-term stability of abandoned waste dumps can be obtained through a survey of existing structures. Also, inspection of similar natural formations may yield useful insight.

We recommend that a program of intermittent inspection of abandoned spoils still exposed be considered. The extent, method and frequency of the program would be determined on the basis of an initial survey.

Aerial photography may offer an economical means of identifying priorities.

The foregoing is not intended to become unduly onerous to operating mines. However, a modest investment of effort using available field examples may prove valuable in designing acceptable valley fills and abandonment plans.

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# TABLE 4.1 SUMMARY OF CREST MONITORING METHODS

| METHOD | GENERIC LOCATION | PURPOSE | USAGE | AVAILABLE DATA: FREQUENCY | AVAILABLE DATA: QUALITY | COMMENTS |
|---|---|---|---|---|---|---|
| VISUAL | CREST PLATFORM | DETECT CHANGES IN DEFORMATION I.E. INCREASING DIFFICULTY IN LEVELLING AND DEVELOPMENT OF STEPS AND CRACKS | ROUTINE TO ALL OPERATIONS | USUALLY LIMITED TO INTERMITTENT REPORT SUMMARIES<br>NOTE: LACK OF RECORDED DATA DOES NOT LESSEN IMMEDIATE VALUE AS AN OPERATING TOOL | VERBAL QUALITATIVE DESCRIPTIONS WITH PRECISE NUMERICAL DEFINITION | EXPERIENCED OPERATIONS PERSONNEL USUALLY TAKE VISUAL SIGNS VERY SERIOUSLY I.E. OCCURRENCE OF UNSTABLE CONDITIONS IS RARELY A SURPRISE |
| | FACE SLOPE | DETECT CHANGES IN FACE DEFORMATION | NOT ROUTINE | OCCASIONAL OBSERVATION OF CREST SCARPS, BULGES, OVERSTEEPENED SEGMENTS | NOTE: OFTEN DIFFICULT TO OBTAIN CLEAR PERSPECTIVE FROM CREST OF ACTUAL PROFILE | |
| WIRELINE EXTENSOMETER | CREST/UPPER FACE SLOPE | MEASURE A COMPONENT OF DEFORMATION AND DETECT ACCELERATION TRENDS SIGNIFYING REDUCTION IN STATE OF STABILITY | ROUTINE TO FOUR OPERATIONS WITH 'ACTIVE' DEFORMING DUMPS | USUALLY 1 OR MORE TIMES PER SHIFT ON ACTIVE DUMPS | VARIABLE TIMING MAY CAUSE ARTIFICIAL PEAKS BUT TREND OVER SEVERAL READINGS IS SATISFACTORY INDICATOR OF CHANGE. ACCORDING READING FREQUENCY IS INCREASED IF ANY READING INDICATES ACCELERATION I.E. (SENSITIVITY) × Fn (ACTIVITY) (RESPONSE TIME) | CONFIGURATION AND POSITIONING HAVE A MAJOR EFFECT ON MAGNITUDE & INTERPRETATION OF DATA |
| | TOE | MEASURE DEFORMATION OF TOE REGION FOR NON-ADVANCING CREST IE. WRAP-AROUND | ONE KNOWN INSTANCE | AT LEAST ONE TIME PER SHIFT | | LINE ATTACHED TO TREE NEAR TOE. NOT PRACTICAL FOR ADVANCING TOE DUE TO BURIAL/IMPACT |
| HYDRAULIC SETTLEMENT GAUGE | CREST | DETECT CHANGES IN ELEVATION AT OUTER CREST WITH RESPECT TO POINT SET BACK FROM CREST | SINGLE TRIAL PERIOD | AS PER ADJACENT WIRELINES | SIMILAR TO WIRELINE EXTENSOMETER | (1) GENERALLY LESS ROBUST AND RELIABLE THAN WIRELINE (2) PROVIDES MORE ACCURATE MEASUREMENT OF CREST SETTLEMENT WHEN DUMP IS VERY ACTIVE. RANGE LIMITED TO 2-3M |
| CONVENTIONAL SURVEY LEVELLING | CREST PLATFORM | DETECT CHANGES IN ELEVATION AT VARIOUS POINTS ON CREST PLATORM | ROUTINE AT 1/2 OPERATIONS | VARIABLE DEPENDING ON VISUAL OBSERVATION | SUBJECT TO FREQUENT DISRUPTION DUE TO LOSS OF PEGS. QUALITY SIMILAR TO WIRELINE | AS (2) ABOVE. OPERATING PERSONNEL NOT INVOLVED AND DO NOT RECEIVE IMMEDIATE FEEDBACK IE. RESPONSE TIME IS LONGER |
| ROTATING LASER BEAM LEVELLING | CREST PLATFORM | AS ABOVE | SINGLE TRIAL PERIOD | COULD BE ROUTINE SEVERAL TIMES PER SHIFT | AS FOR WIRELINE | AFFECTED BY FOG. OPERATING PERSONNEL CAN BE INVOLVED. IMPRACTICAL FOR RAMPING CREST. ? WINTER OPERATIONS |
| CONVENTIONAL GEODETIC SURVEY | CREST PLATFORM | DETECT CHANGES IN MAGNITUDE AND DIRECTION OF DISPLACEMENTS AT VARIOUS POINTS ON CREST PLATFORM | 3 OPERATIONS | FREQUENCY DEP. ON ACTIVITY - UP TO SEVERAL TIMES PER SHIFT IF HIGHLY ACTIVE | HIGHEST | PROVIDES COMPLETE NUMERICAL CHARACTERIZATION OF MOVEMENT. EXPENSIVE IN MAN-POWER AND DOES NOT INVOLVE OPERATING PERSONNEL |
| | FACE SLOPE | AS ABOVE FOR POINTS ON FACE | 1 OPERATION (INACTIVE DUMP) | ? WEEKLY TO MONTHLY | HIGHEST | AS ABOVE. PROVIDES USEFUL DATA ON SPATIAL VARIATION OF MAGNITUDE AND DIRECTION OF DEFORMATION |
| PIEZOMETER | FOUNDATION BENEATH TOE | DETECT AND MEASURE CHANGES IN FOUNDATION PORE FLUID PRESSURE. ALSO CRUDE INCLINOMETER | | | ADEQUATE IF UNDAMAGED AND PROPERLY INSTALLED | READILY DAMAGED AND IMPRACTICAL TO REPLACE. DIFFICULT TO MONITOR IF REMOTE AND WINTER. IMPRACTICABLE TO RELIABLY PREDICT SAFETY BASED ON SPECIFIC VALUES MEASURED. BETTER INDICATION IS MASS DEFORMATION |

TABLE 4.2 SUMMARY OF AVAILABLE MONITORING DATA

| MINE SITE CODE | PERIOD OF DATA COLLECTION | TYPE OF MONITORING USED | FREQUENCY |
|---|---|---|---|
| A | 1984 - | WL1 | SHIFT + |
| B | 1981 - | WL2 | SHIFT + |
| | | H | TRIAL |
| | | L | TRIAL |
| C | 1970 - | WL2 | SHIFT + |
| E | 1970 - | L | SHIFT + AS REQUIRED, BASED ON VISUAL |
| D | 1985 - | WL1 | SHIFT AS REQUIRED, BASED ON VISUAL |
| F | 1984 - | S | INTERMITTENT; AS REQUIRED |
| G | 1983 - | S | DAILY AS REQUIRED |
| H | 1983 - | S | INTERMITTENT; AS REQUIRED |

| | |
|---|---|
| WL1 | SINGLE STAND WIRELINE EXTENSOMETER |
| WL2 | TWO STAND WIRELINE EXTENSOMETER |
| H | HYDRAULIC LEVEL U-TUBE |
| L | PRECISE SURVEY LEVELLING |
| S | PRECISE SURVEY OF TOTAL MOVEMENT |
| P | PIEZOMETERS |

TABLE 4.3 ROCK TYPE AND STRENGTH

| MATERIAL TYPE | RANGE OF AVERAGE UNCONFINED COMPRESSIVE STRENGTH (MPa) | | |
|---|---|---|---|
| Sandstone | 110 | to | 170 |
| Siltstone | 60 | to | 110 |
| Mudstone/Shale | 30 | to | 80 |

| PROPERTY CODE | ESTIMATED PROPORTION SAND STONE | SILT STONE | MUDSTONE/ SHALE | WEIGHTED AVERAGE UNCONFINED COMPRESSIVE STRENGTH (MPa) | | |
|---|---|---|---|---|---|---|
| A | Relatively weak materials | | | (50 | to | 80) |
| B | Relatively stromg | | | (100 | to | 150) |
| C | 40 | 40 | 20 | 74 | to | 128 |
| D | Relatively weak | | | (50 | to | 80) |
| E | 55 | 25 | 20 | 82 | to | 137 |
| F | 50 | 30 | 20 | 79 | to | 134 |
| G | 20 | 40 | 40 | 58 | to | 110 |
| | | | RANGE | 50 | to | 150 |

Note : Ranges and averages are approximate , based on available data. Wide variations are to be expected at individual mine sites.

TABLE 5.1 FAILURE FOUNDATION PROFILE TYPES

| PROFILE TYPE (*) | NUMBER OF FAILURES | PER CENT PROFILE TOTAL |
|---|---|---|
| A | 35 | 65 |
| B or AB | 9 | 17 |
| BA or ABA or BC | 6 | 10 |
| C or ABC | 2 | 4 |
| UNKNOWN | 2 | 4 |
| TOTAL | 54 | |

A = Concave downwards

B = Convex

C = Planar

Note * : See figure 4.4 for profile types

TABLE 5.2 FACTORS AFFECTING FAILURES

| SLOPE RANGE | CONTRIBUTING FACTOR | NUMBER OF EVENTS | PER CENT OF TOTAL |
|---|---|---|---|
| greater than 24 deg | SPRING THAW | 13 | 36 |
| | RAIN STORM EVENT | 11 | 28 |
| | UNCLEAR | 8 | 22 |
| | POOR ROCK QUALTIY | 6 | 17 |
| | TOE STEEPENING (TYPE B PROFILE) | 4 | 11 |
| | FOUNDATION CONDITIONS | 3 | 8 |
| | (Number of events = 36) | | |
| less than 24 deg | POOR ROCK QUALITY | 4 | 44 |
| | RAIN EVENT | 3 | 33 |
| | TOE STEEPENING | 3 | 33 |
| | UNCLEAR | 3 | 33 |
| | SPRING THAW | 1 | 11 |
| | FOUDATION CONDITIONS | 1 | 11 |
| | (Number of events = 9) | | |

Note : Some events are related to more than one factor

TABLE 5.3 SUMMARY OF FAILURES RELATED TO FOUNDATION CONDITIONS

| EVENT NUMBER | PROPERTY CODE | MONTH | TOE SLOPE | IDENTIFIED OR SUSPECTED CAUSE |
|---|---|---|---|---|
| 48 | A | June | 14 | slide debris |
| 49 | A | June | 16 | slide debris |
| 24,27 | C | Nov | <29 | saturated surficial organics |
| 17 | C | Jun | 15 | ditto |
| 9 | C | Jan | 11 | incorporated snow and/or frozen foundation |
| 149 | C | July | <20 | frozen foundation with seepage |
| 76 | D | May | 13 | seepage and fine waste rock |
| 79 | D | June | 18 | ditto |
| 81 | D | march | 18 | ditto |

BREAKDOWN BY FACTOR

| CAUSE TYPE | NUMBER OF EVENTS |
|---|---|
| Saturated, unconsolidated surficial foundation soils | 3 + numerous other cases of localised toe spreading |
| Foundation seepage | |
| - compounded by poorly drained fine waste rock | 3 |
| - compunded by ground freezing | 2 |
| Presence of previous failure debris | 2 |

TABLE 5.4 MAGNITUDE AND DIRECTION OF CREST DEFORMATION

| SITE CODE | DUMP HT (M) | TOE SLOPE (DEG) | VECTOR DIP RANGE | AVERAGE VECTOR DIP (DEG) | RATE NEAR CREST (M/D) | SET-BACK DIST. (M) | RATE BEHIND CREST (M/D) | SET-BACK DIST. (M) | TREND |
|---|---|---|---|---|---|---|---|---|---|
| B | 200 | 10-20 | | 53 | 0.63 | near | 0.19 | 30 | |
| | | | | 46 | 0.76 | near | 0.34 | 30 | |
| | | | | 46 | 0.76 | near | 0.17 | 30 | |
| | | | | 46 | 0.17 | near | 0.09 | 30 | |
| C | 120 | | | 55 | | | | | See fig 5.41 |
| D | | | 54-59 | 58 | 0.31 | 5 | 0.09 | 25 | |
| | | | 46-53 | 49 | .26 | 5 | 0.05 | 25 | |
| | | | 55-62 | 59 | 0.18 | 5 | 0.04 | 25 | |
| | | | 58-67 | 62 | 0.07 | 5 | 0.03 | 25 | Dip decreases from crest |
| E | 130 | 22 | 49-60 | 55 | 2.93 | 1 | 1.212 | 10 | |
| | 130 | 28 | 51-53 | 52 | 3.55 | 1 | 2.25 | 10 | |
| | 130 | 30 | 44-58 | 51 | 3.89 | 1 | 0.83 | 10 | |
| | 190 | 10 | 50-64 | 57 | 2.83 | 1 | 0.14 | 10 | |
| | 190 | 10 | 50-58 | 54 | 2.93 | 1 | 1.57 | 10 | |
| | 190 | 10 | 49-59 | 54 | 1.66 | 1 | 0.47 | 10 | |
| F | 65 | 26 | 38-48 | | | | | | Monitoring of prisms on crest for 1.5 years after dump complete |
| | 40 | 0 | 56-60 | | | | | | |
| | 80 | 17 | | 54 | | | | | |
| G | 60 | 30 | 45-55 | 50 | | | | | |
| | 76 | 20 | 50-55 | | | | | | Near crest |
| | 76 | 20 | 25-35 | | | | | | Near toe |
| H | 230 | 33 | 33 | | | | | | Near crest |
| | | | 23 | | | | | | Near toe |

Dip Angle Range : 38 - 67 at/behind crest. Average = 53 deg.
23 - 35 near toe

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TABLE 5.5 SUMMARY OF FAILURE MECHANISMS

| EVENT NO | TYPE | MONTH | TOE SLOPE (DEG) | DUMP HEIGHT (M) | PROFILE TYPE | SCARP SETBACK (M) | SCARP WIDTH (M) | FAILURE PLANE GEOMETRY |
|---|---|---|---|---|---|---|---|---|
| 62 | RAPID | JULY | 34 | 140 | A | 40 | 80 | Narrow platform on steep slope. Final scarp steepens to 70 deg through upper 30m. |
| 1 | RAPID | JUNE | 28-34 | 200 | A | 15 | 65 | Narrow platform on steep terrain in gully - failure mechanism similar to above |
| 3 | RAPID | SEPT | 26-28 | 220 | A | 5-10 | 170 | As above |
| 5 | RAPID | JULY | 20-26 | 210 | A | Nil | 100 | As above, scarp does not propogate up to/behind crest. |
| 12 | RAPID | AUG | 35+ | 180 | C | 5-10 | | Inactive.Snow causes failure plane exits 50m above toe. |
| 20 | RAPID | MAY | 30 | 130 | B | 10 | | Lower 100m passes along steep fndn Upper 30m curves to 60deg. |
| 112 | RAPID | AUG | 20 | 240 | A | 50 | 250 | Series of regressive scarps. |
| 113 | RAPID | OCT | 21 | 250 | A | 50 | 220 | Site of previous failure. Upper 75m steepens to 50+deg |
| 54 | RAPID | SEPT | 22+ | 220 | A/B | 45 | 300-400 | Steep segment near toe |
| 48 | RAPID | JUNE | 14-26 | 200 | A/B | 57 | 140 | Lower 70m of face steepens to 40deg. |
| 49 | RAPID | JUNE | 16 | 180 | A/B | 67 | 200 | 3 previous failures.Upper 20m steepens to 60 ,deg. |
| 76 | RAPID | MAY | 13 | 170 | A | 10Init. 50Fin. | 250 | Fine waste and seepage.Upper 50m at 50-70deg immed. after and flattens as crest regresses. |
| 75 | RAPID | MAR | 27 | 160 | A | 10 | | As above. |
| G | CREEP | | 20 | 64 | A | | | Toe slope displayed 13m parallel to natural slope as fndn spreads. |
| 149 | RAPID | JULY | 20 | 250 | A | 100 | 100 | Hydrostatic pressure on frozen fndn Failure extends across width of narrow platform. |
| 9 | CREEP | JAN | 11 | 260 | A | 16 | 220 | Crest slumped 70m exposing 46deg scarp.?Seepage on frozen fndn. |

TABLE 5.6 OBSERVED FAILURE EVENTS

| SITE CODE | EVENT NO | DUMP HEIGHT (M) | TOE SLOPE (DEG) | FAILURE TYPE | KEY ELEMENTS OF EYEWITNESS OBSERVATIONS |
|---|---|---|---|---|---|
| A | 112 | 240 | 20 | RAPID | "series of regressive failures" with "3 or 4 separate slides" |
| A | 49 | 180 | 16 | RAPID | failure started with "bottom of ... dump exploding into the valley". Waste materail to flow like fluid. |
| B | 134 | 220 | 15 | SLIVER | visual signs of large crest deformation observed 2 days prior caused dump shutdown. |
| C | 12 | 180 | 20 | RAPID | Visual and wireline gave ample warning that failure impending. Snow observed. |
| C | | 200 | 25+ | RAPID | Failure initiated in toe region, scarp propogating up face to crest. |
| D | | 120-170 | 20-30 | RAPID | Unusually rapid loss of crest |

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TABLE 6.1 ROCKFILL TEST DATA

| MATERIAL | MAX. SIZE (MM) | d10 SIZE (MM) | d50 SIZE (MM) | Cu | void ratio | MEASURED $\phi_o$ | MEASURED $\Delta\phi$ | U.C.S. (MPa) | $\phi_{basic}$ | PREDICTED $\phi_o$ | PREDICTED $\Delta\phi$ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| El Infiernillo diorite | 200-220 | 20 | 75 | 5 | 0.56 | 46 | 9 | | | | |
| Pyramid Dam Argillite | 150 | 6 | 45 | 10 | 0.45 | 49 | 9 | 109 | 29 | 46 | 7 |
| Oroville Dam Amphibolite | 150 | 1.4 | 35 | 32 | 0.22 | 49 | 7 | 197 | 28 | 52 | 9 |
| Durable Sandstone | 300 | 2 | 90 | 60 | 0.24 | 46 | 5 | 35 | 28 | 48 | 10 |
| Non-durable Shale | 300 | 40 | 180 | 5 | 0.45 | 49 | 9 | 20-60 | 33 | 47 | 7 |

Notes: $\phi_o$ is angle of internal friction at low confining stress.
$\Delta\phi$ is reduction of angle of internal friction for each log cycle of confining stress.

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TABLE 7.1 SAMPLES FROM CREST AT SITE C

| SAMPLE NUMBER | DRY DENSITY (KG/M3) | VOID RATIO | REMARKS |
|---|---|---|---|
| 1 | 1490 | 0.48 | Frozen,high moisture content |
| 2 | 954 | 1.32 | ditto |
| 2B | 1691 | 0.56 | ditto |
| 3(TOTAL) | 1917 | 0.26 | Remained intact |
| 3(Chunk 1) | 1828 | 0.18 | } |
| 3(Chunk 2) | 1793 | 0.20 | }chunks exhibit apparent cohesion |
| 3(Chunk 3) | 1930 | 0.12 | } |

TABLE 7.2 EVALUATION OF SHEAR STRENGTH PARAMETERS WITHIN DUMPS

| LOCATION WITHIN DUMP | ROCKFILL MATRIX GRADATION | d50 (mm) | POSSIBLE VOID RATIO | ASSUMED PARTICLE STRENGTH (MPa) | PREDICTED ANGLE OF INTERNAL FRICTION UNDER NORMAL STRESS EQUIVALENT TO 5M (deg) | 50M (deg) | 200M (deg) |
|---|---|---|---|---|---|---|---|
| Dump Base | Segregated Coarse | 200 to 800 | 0.3 | 120 | 49 | 41 | 36 |
| | | | 0.4 | | 46 | 39 | 35 |
| | | | 0.6 | | 42 | 36 | N/A |
| | | | 1.0 | | 39 | | |
| Intermediate Elevation | Well graded intermediate gradation | 50 to 200 | 0.2 | 100 | 55 | 44 | 38 |
| | | | 0.4 | | 46 | 38 | 34 |
| | | | 0.6 | | 41 | 36 | N/A |
| Near Crest | Well graded finer end of gradation spectrum | 20 to 100 | 0.2 | 75 | 54 | 43 | N/A |
| | | | 0.4 | | 45 | 38 | N/A |
| | | | 0.6 | | 41 | 35 | N/A |

Notes :

1. Method of prediction by Barton & Kjaernsli(1981) assuming angular material with basic friction angle = 28 deg

2. Predicted friction angle not sensitive to value of mean particle size, moderately sensitive to particle strength, and most sensitive to void ratio.

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# Volume

# 2

**Golder Associates**
CONSULTING GEOTECHNICAL AND MINING ENGINEERS

CONTRACT REPORT NO. 69188-01-SQ

# REGIONAL STUDY OF COAL MINE WASTE DUMPS IN BRITISH COLUMBIA

## STAGE II

## VOLUME II
## FIGURES AND APPENDICES

144 pp

November, 1987

862-1231

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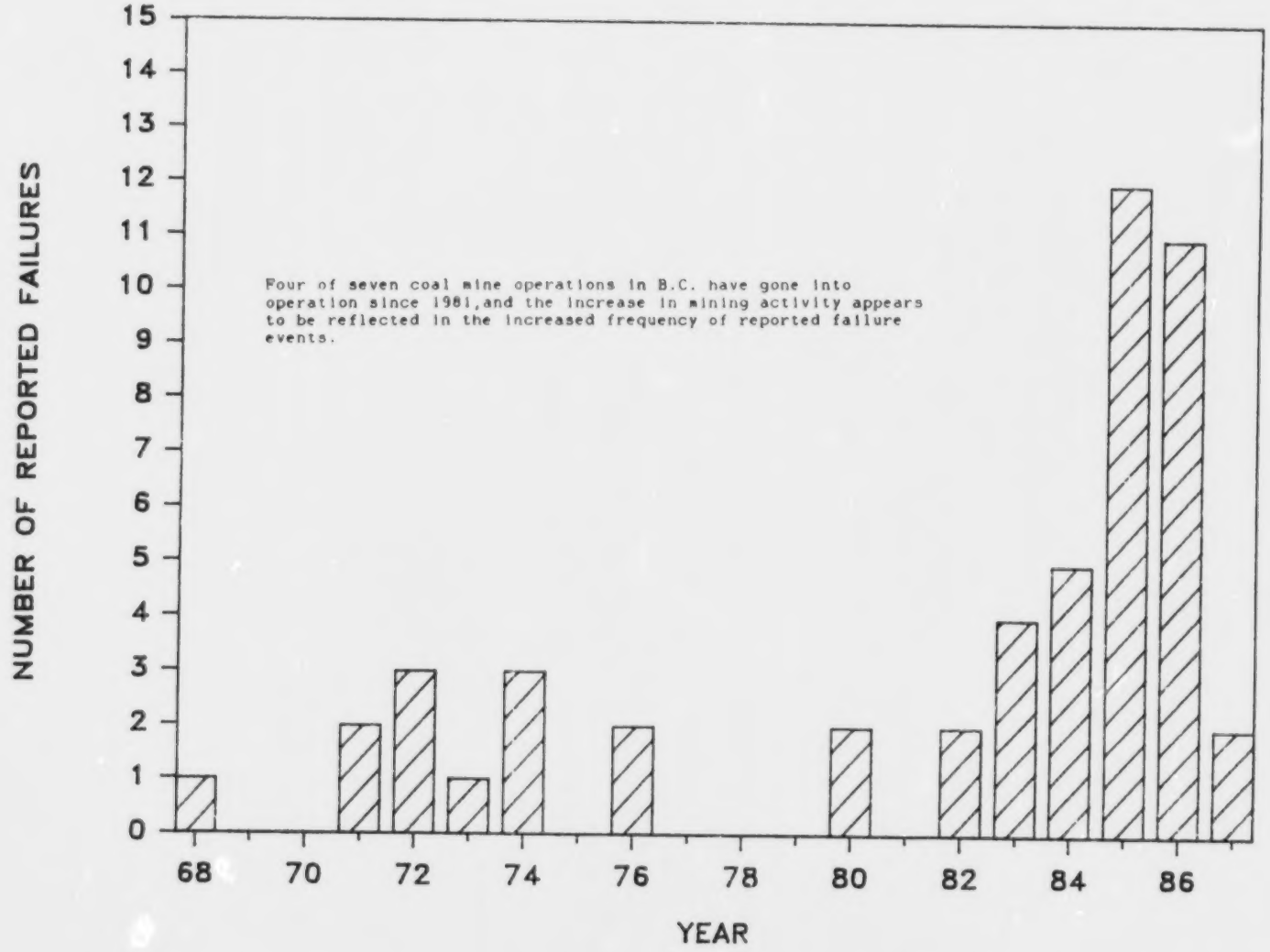


NUMBER OF REPORTED FAILURES BY YEAR

Figure 1.1

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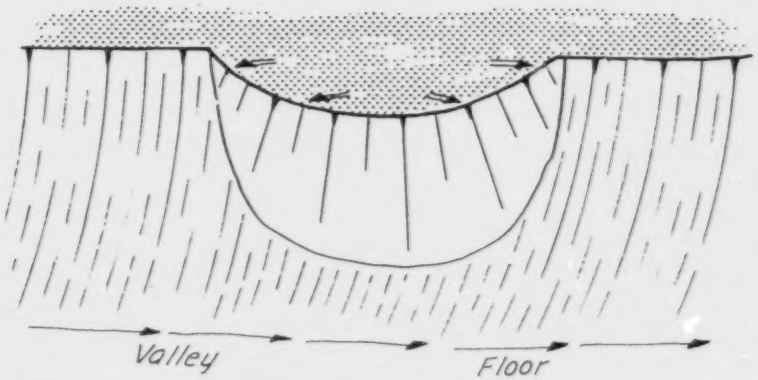
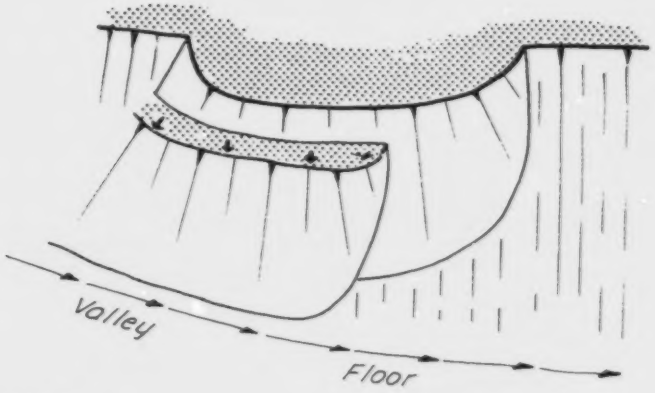
# TYPICAL WASTE DUMP CONFIGURATIONS ON MOUNTAIN TERRAIN

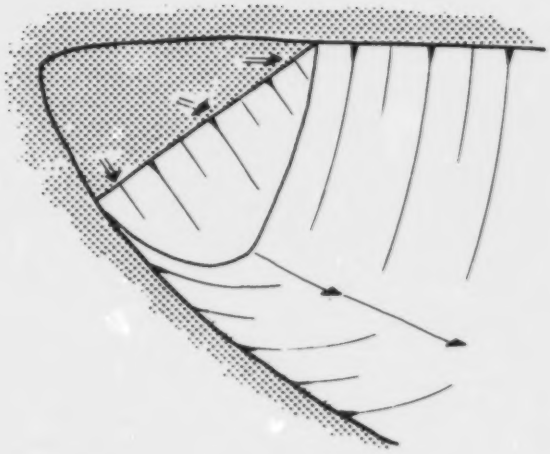
Figure **4.1**




**a) SIDEHILL FILL**




**b) WRAP-AROUND**



**c) VALLEY FILL**

→ *Signifies direction of crest advance*

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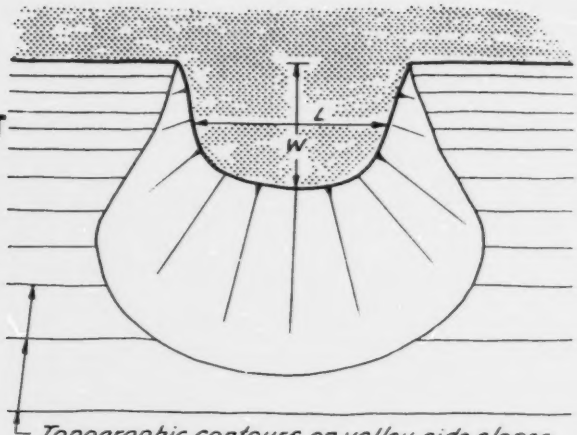
# TYPICAL WASTE DUMP CREST SHAPES

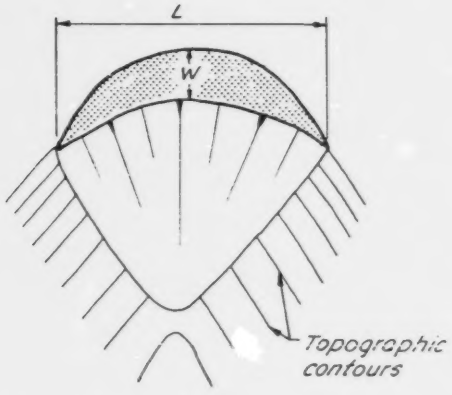
Figure **4.2**

**a) CONVEX CREST**



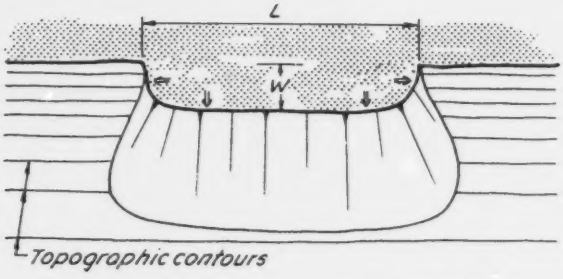

Relatively Narrow Platform : W may be > L

**b) CONCAVE CREST**




Large Gullies on Steep Terrain
L >> W

**c) PLANAR CREST**




Typical Sidehill Dump L ≥ W

⇐ potential direction of crest advance

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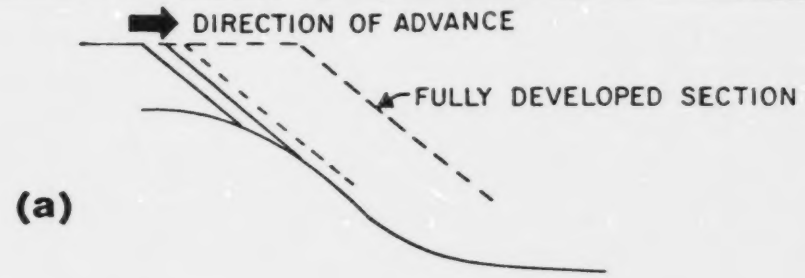
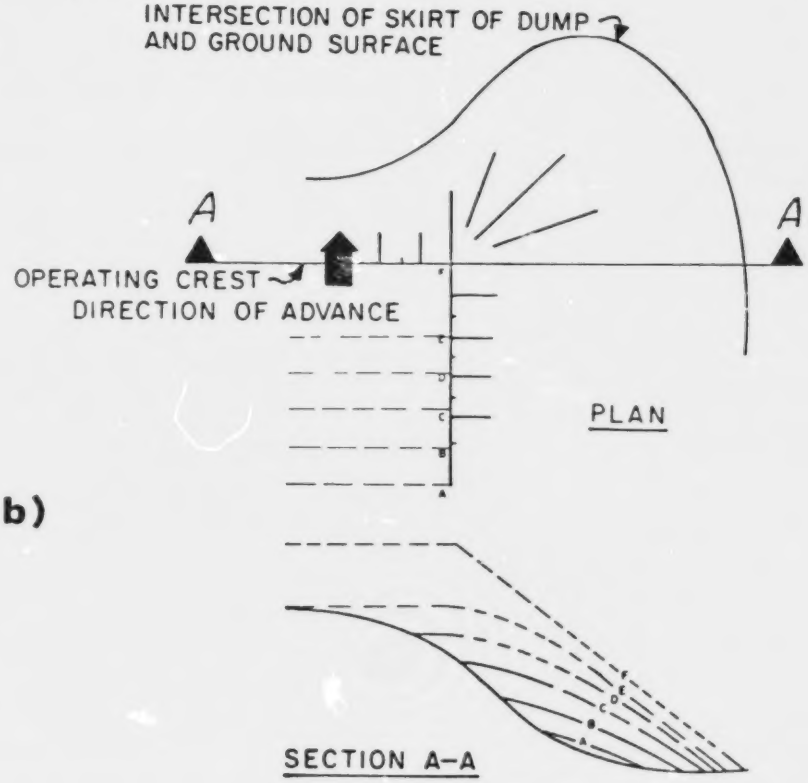
# DIRECTION OF ADVANCE OF DUMP RELATIVE TO TOPOGRAPHIC SLOPE

Figure **4.3**




(a)




(b)

Waste rock dump is developed on topographic slope that has S-shaped fall-line profile.

In case (a) above, dump face is advanced in direction parallel to fall-line. When toe of dump advances onto steep segment of slope, failure occurs, and fully developed section cannot be achieved.

In case (b) above, the dump is developed in a direction paralled to the topographic contours, while maintaining the operating crest perpendicular to the direction of advance. The profiles marked A, B, ...F on section A-A represent the dump surface when the crest is at the positions marked A, B, ...F respectively on the plan. As illustrated, waste rock is in place above the gentle topographic slopes below the base region of the dump, and provides lateral support before loads are applied above the steep segments of the topographic slope. As a result, the section remains stable throughout all stages of development.

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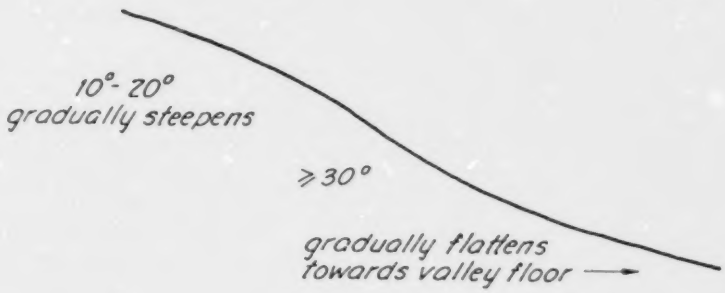
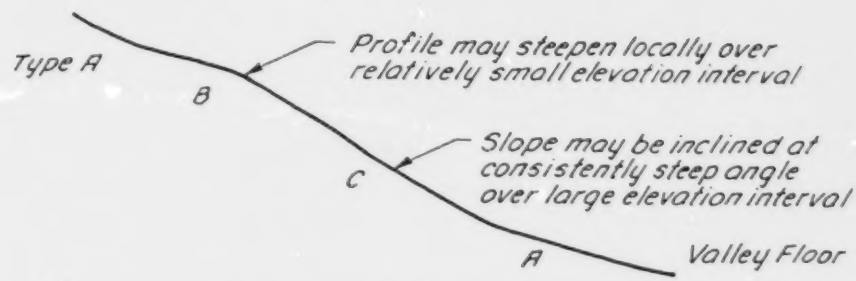
# TYPICAL CROSS SECTIONS OF NATURAL GROUND SURFACE

Figure **4.4**

**TYPE A - CONCAVE**




**TYPE B - CONVEX**




**TYPE C - SLOPE CONSISTENTLY STEEP OVER LARGE ELEVATION INTERVAL**

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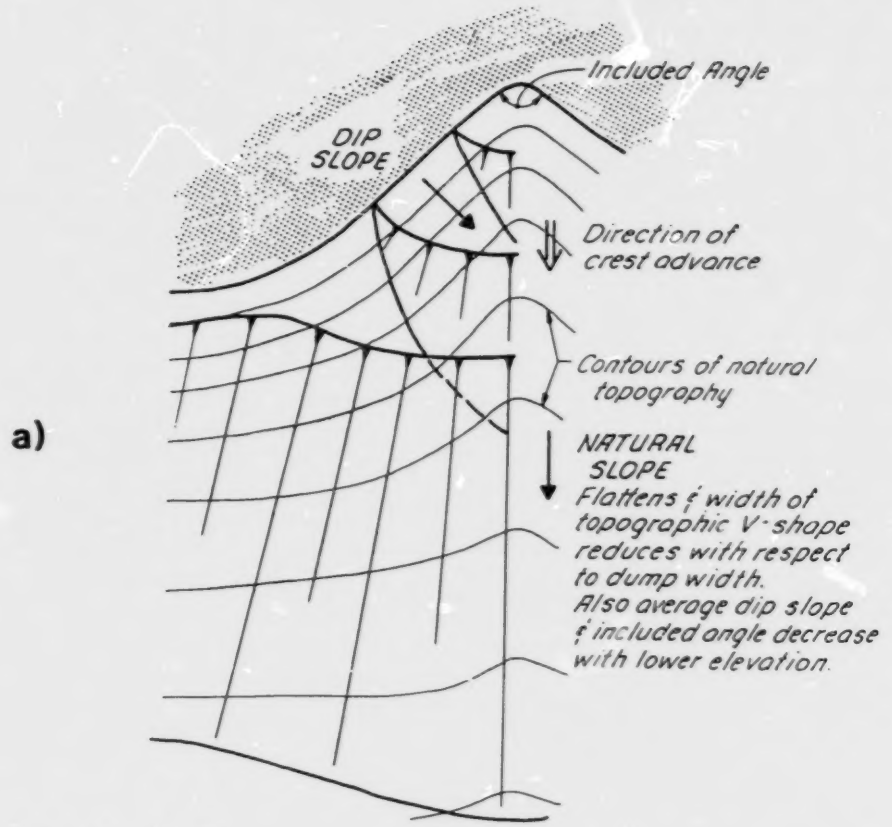
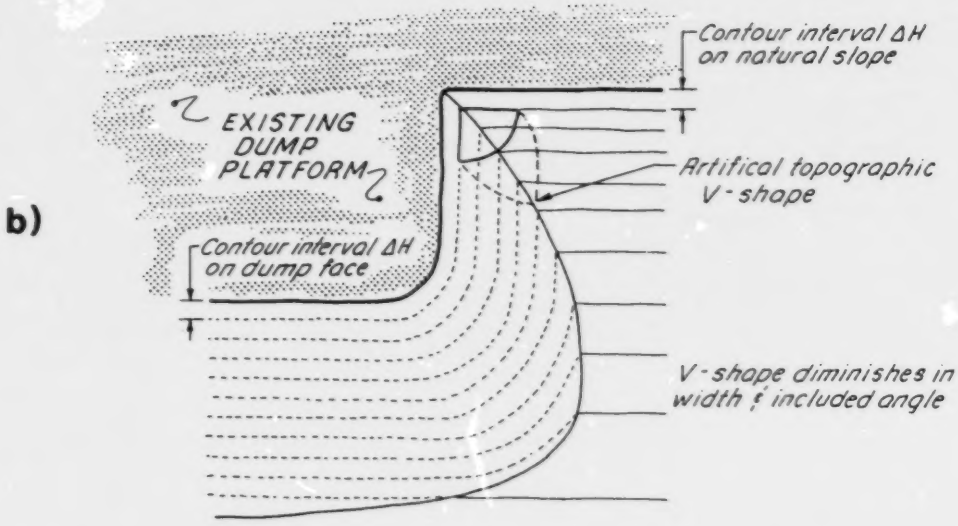
## TOPOGRAPHIC V- SHAPES

Figure **4.5**


a)
Included Angle
DIP SLOPE
Direction of crest advance
Contours of natural topography
NATURAL SLOPE
Flattens & width of topographic V-shape reduces with respect to dump width.
Also average dip slope & included angle decrease with lower elevation.


b)
Contour interval ΔH on natural slope
EXISTING DUMP PLATFORM
Artifical topographic V-shape
Contour interval ΔH on dump face
V-shape diminishes in width & included angle

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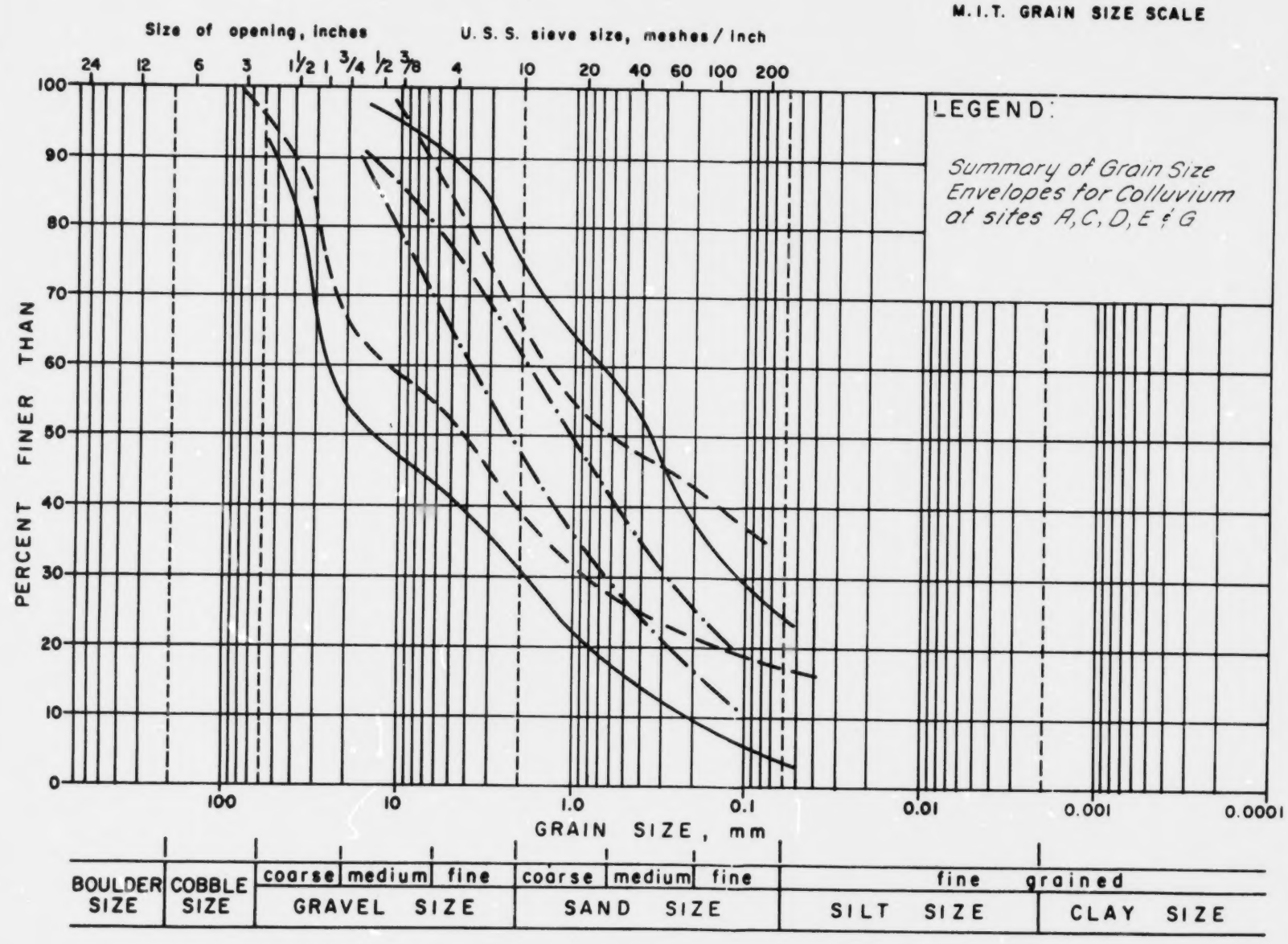
PROJECT NO. 867-1231 DRAWN BAD REVIEWED DATE May '87


M.I.T. GRAIN SIZE SCALE
Size of opening, inches
U. S. S. sieve size, meshes / inch
24 12 6 3 1½ 1 ¾ ½ ⅜ 4 10 20 40 60 100 200
LEGEND:
Summary of Grain Size Envelopes for Colluvium at sites A, C, D, E & G
PERCENT FINER THAN
100 90 80 70 60 50 40 30 20 10 0
100 10 1.0 0.1 0.01 0.001 0.0001
GRAIN SIZE, mm
BOULDER SIZE
COBBLE SIZE
coarse medium fine
GRAVEL SIZE
coarse medium fine
SAND SIZE
fine grained
SILT SIZE
CLAY SIZE

GRAIN SIZE DISTRIBUTION
COLLUVIUM

Figure 4.6(a)

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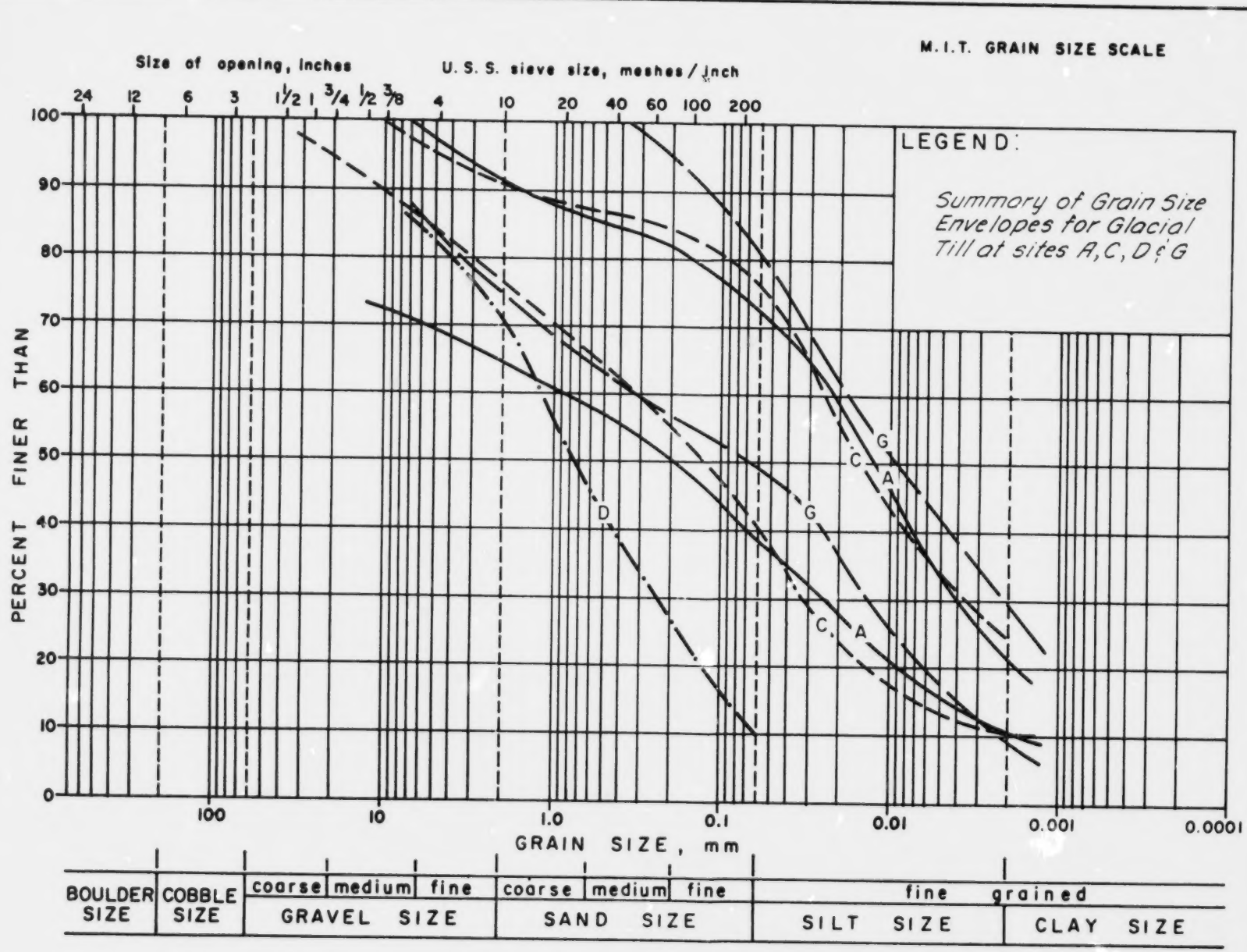


GRAIN SIZE DISTRIBUTION
GLACIAL TILL

Figure 4.6(b)

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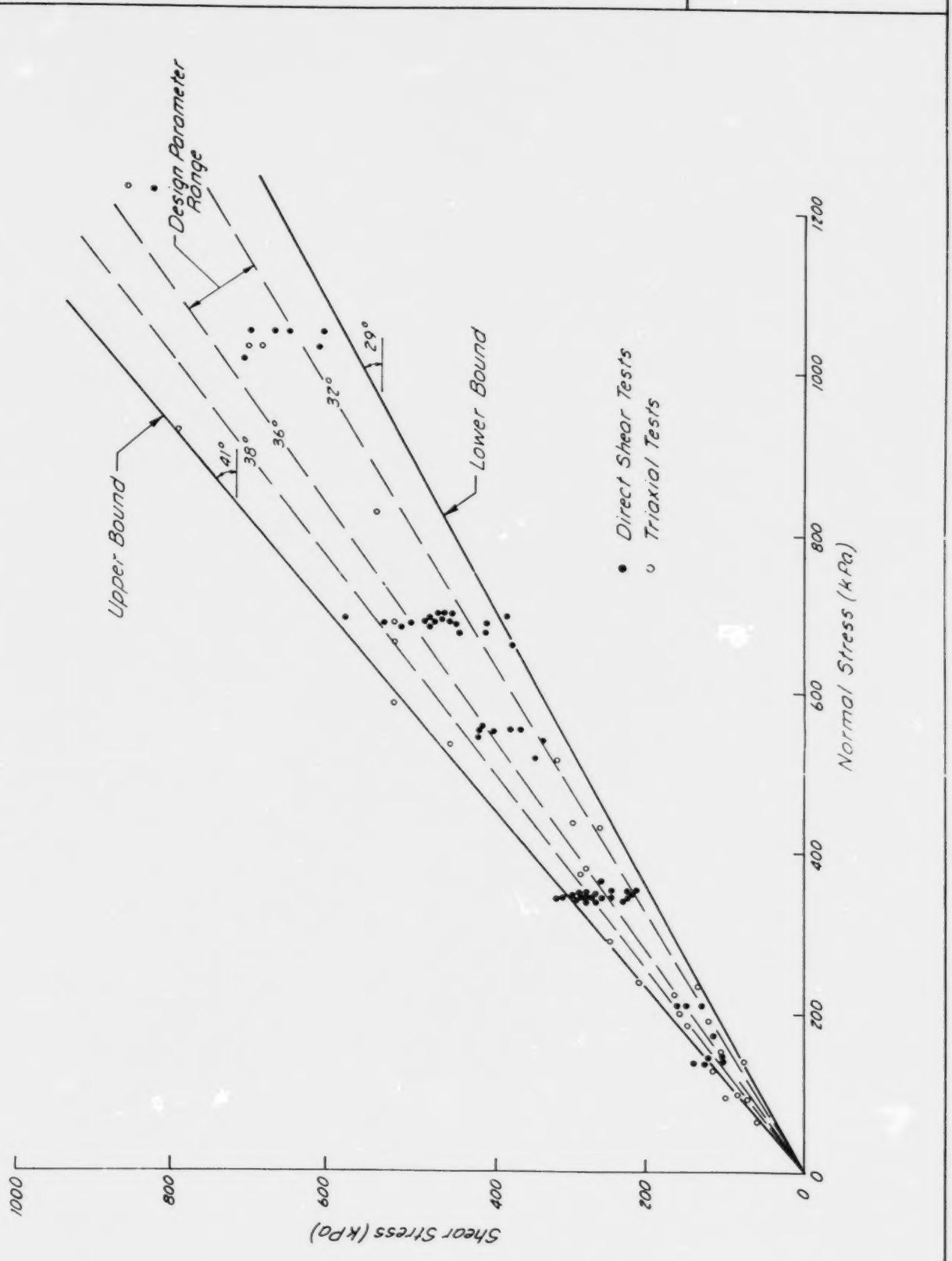
**SUMMARY OF SHEAR STRENGTH TESTS FOR COLLUVIUM & GLACIAL TILL**

Figure **4.7**




PROJECT NO. 862-1231 DRAWN BAD REVIEWED ..... DATE MAY '87

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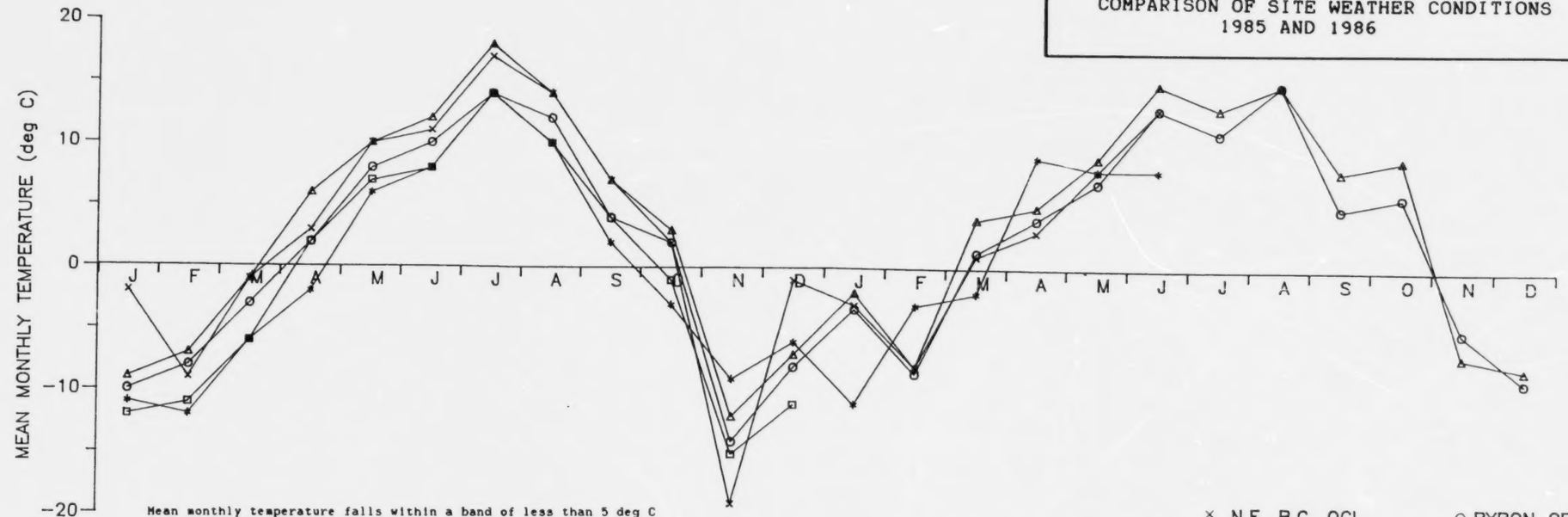
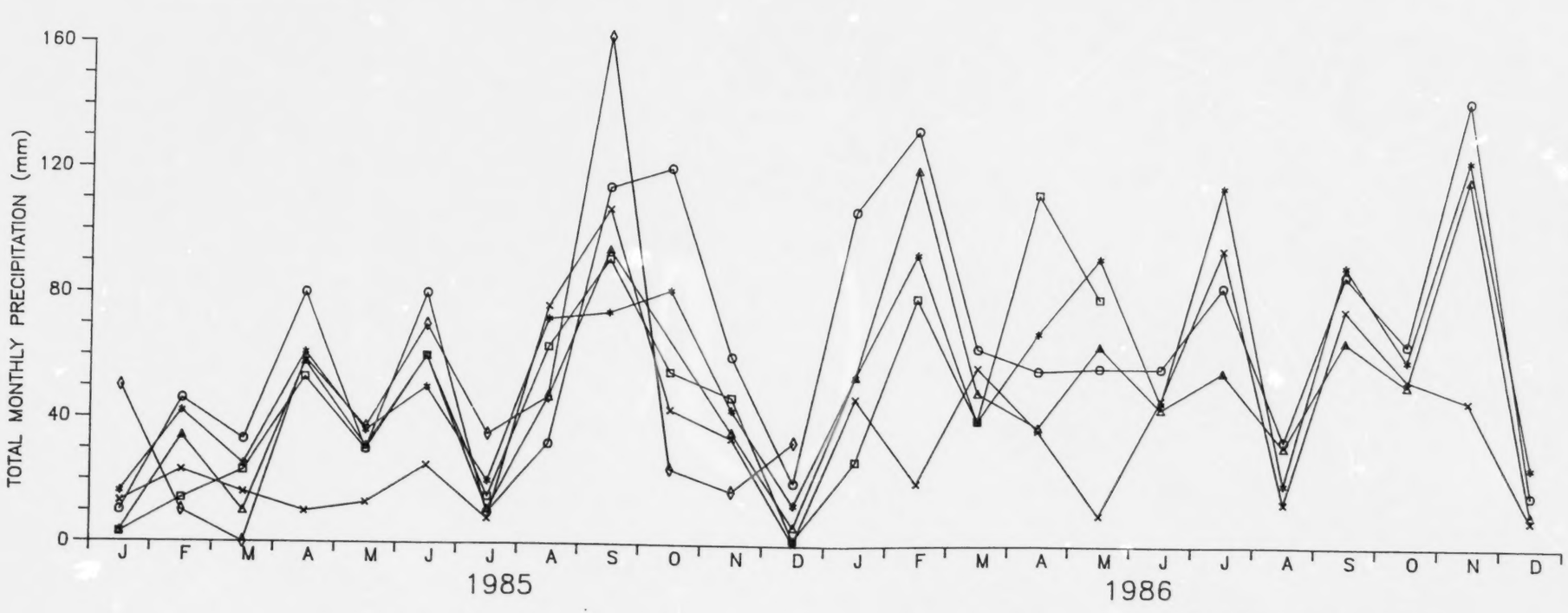
COMPARISON OF SITE WEATHER CONDITIONS
1985 AND 1986

**Figure** 4.8


MEAN MONTHLY TEMPERATURE (deg C)
20
10
0
-10
-20
J F M A M J J A S O N D J F M A M J J A S O N D
Mean monthly temperature falls within a band of less than 5 deg C for all sites. Sharp drop in temperature in November 1985 may have resulted in failure due hydrostatic pressure under frozen ground in the following summer. Total monthly precipitation shows broader variation but trends in the N.E. and S.E. of B.C. are consistent in each area.
× N.E. B.C. QCL
◊ SMOKEY RIVER
△ SPARWOOD
○ BYRON CR.
✱ FORDING
□ LINE CR.




PROJECT NO. ..... DRAWN ..... REVIEWED ..... DATE .....

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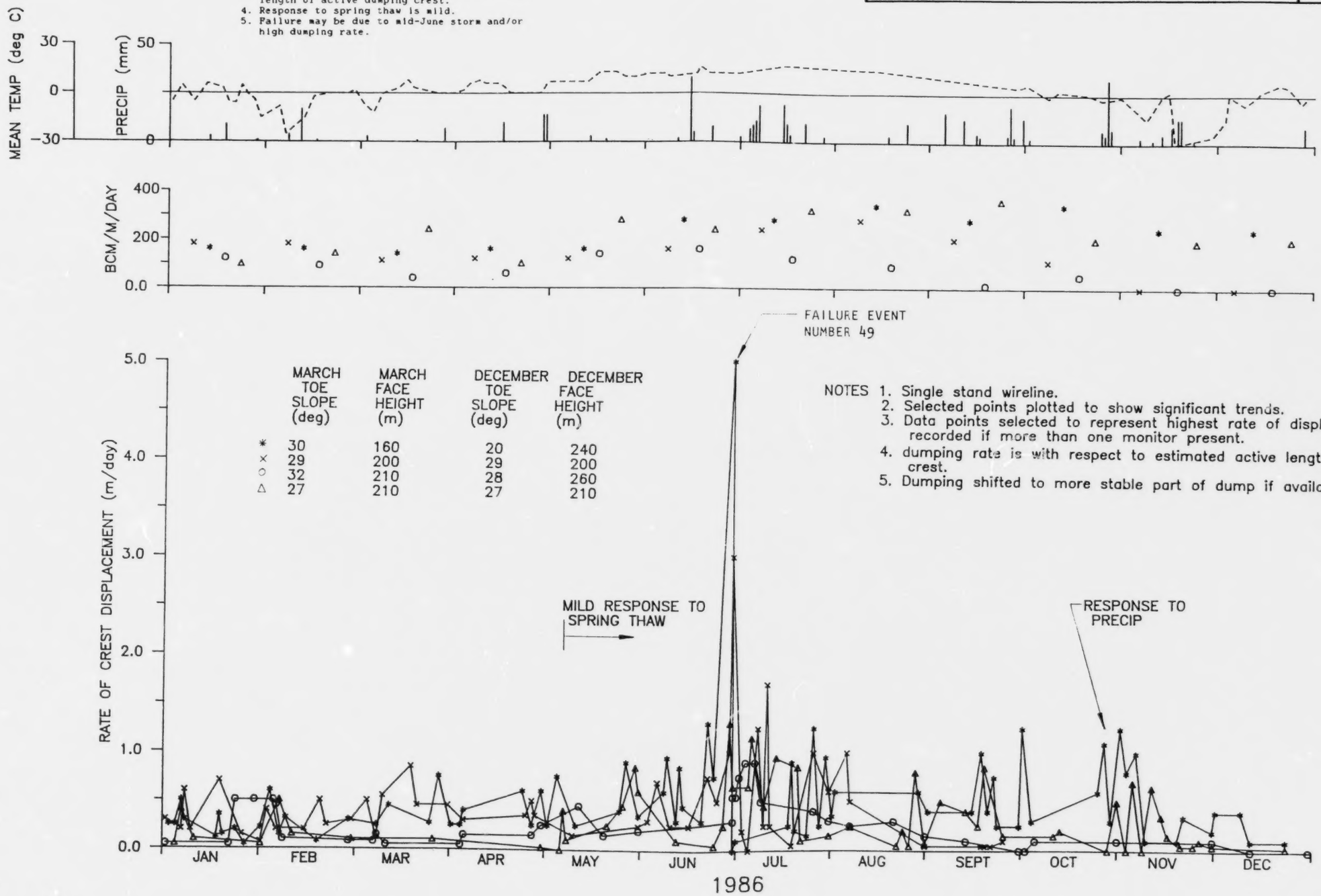
Notes :
1. Single stand wireline anchored to crest pin
2. Selected points plotted for monitors showing highest reading on dump
3. Rate of dumping is with respect to estimated length of active dumping crest.
4. Response to spring thaw is mild.
5. Failure may be due to mid-June storm and/or high dumping rate.

SITE A DUMPS : GROUP A
1986

**Figure** 5.1




NOTES 1. Single stand wireline.
2. Selected points plotted to show significant trends.
3. Data points selected to represent highest rate of displacement recorded if more than one monitor present.
4. dumping rate is with respect to estimated active length of crest.
5. Dumping shifted to more stable part of dump if available.

PROJECT NO. ..... DRAWN ..... REVIEWED ..... DATE .....

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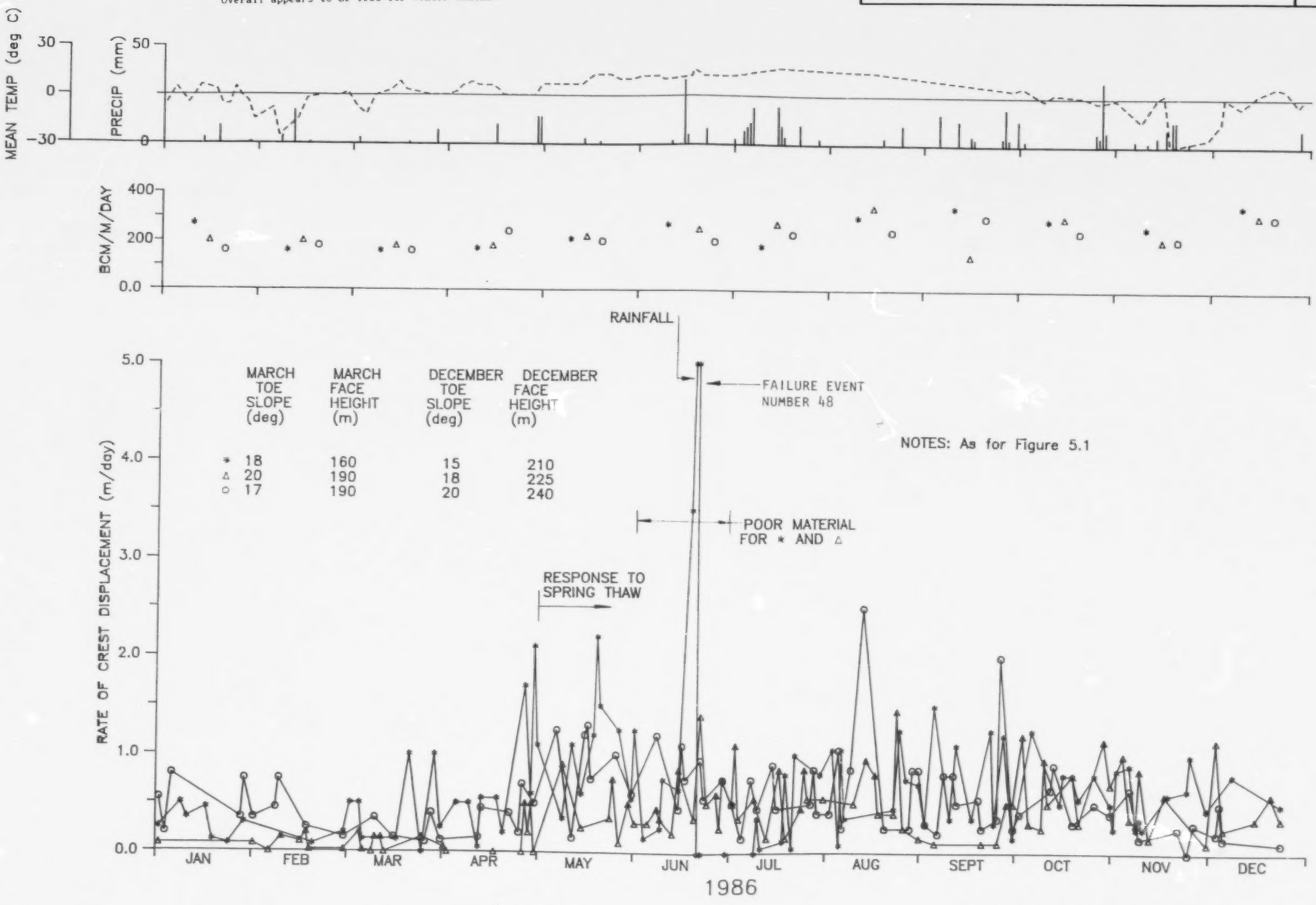
Response to spring thaw is more marked than for group A dumps.
June failure may be due to storm and/or abnormally fine grained material.
Overall appears to be less for winter months.

SITE A GROUP B DUMPS
1986

**Figure** 5.2




PROJECT NO. ..... DRAWN ..... REVIEWED ..... DATE .....

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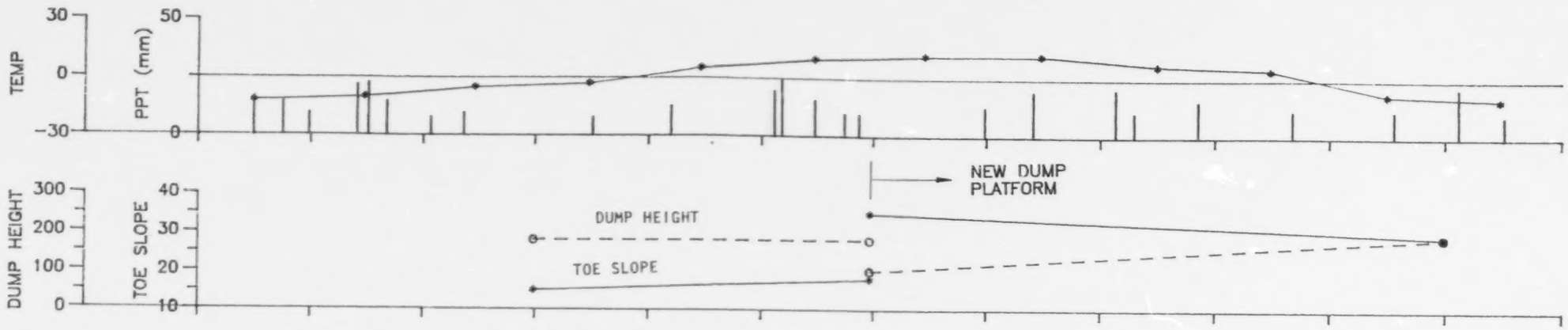
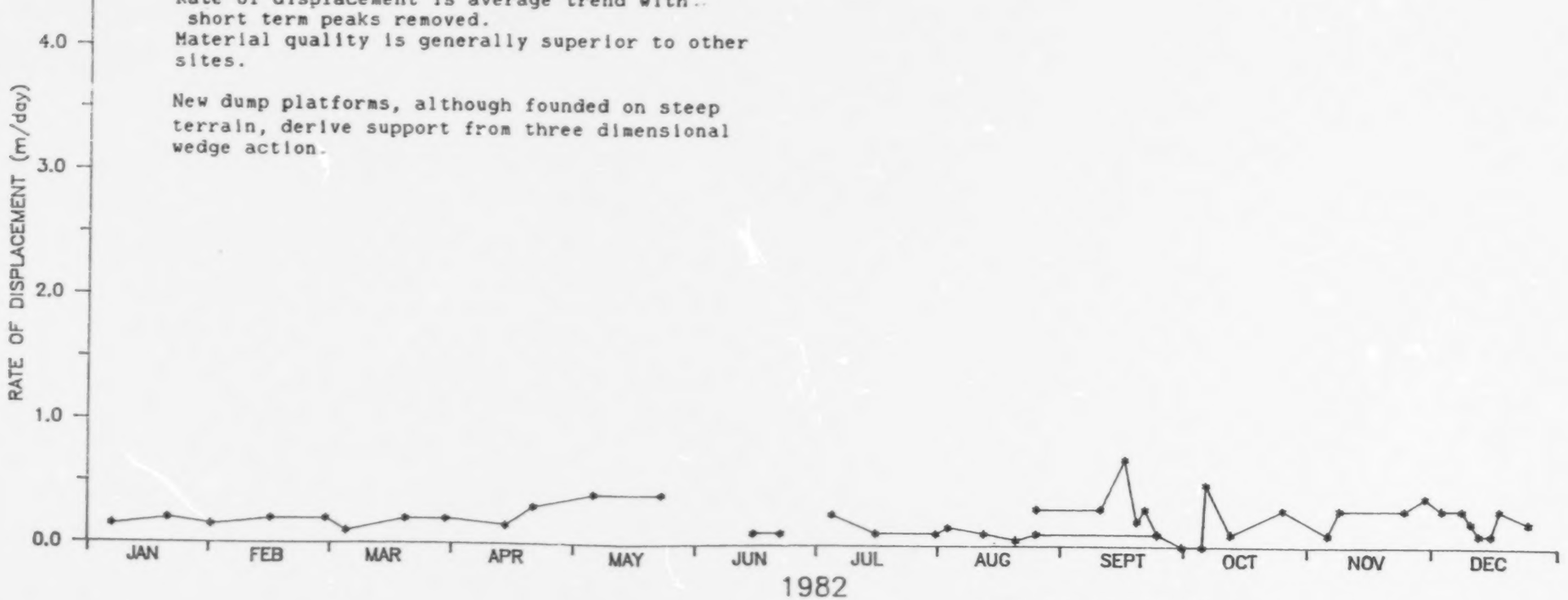
SITE B : YEAR ONE

**Figure** 5.3




Rate of displacement is average trend with short term peaks removed.
Material quality is generally superior to other sites.

New dump platforms, although founded on steep terrain, derive support from three dimensional wedge action.




PROJECT NO. ..... DRAWN ..... REVIEWED ..... DATE .....

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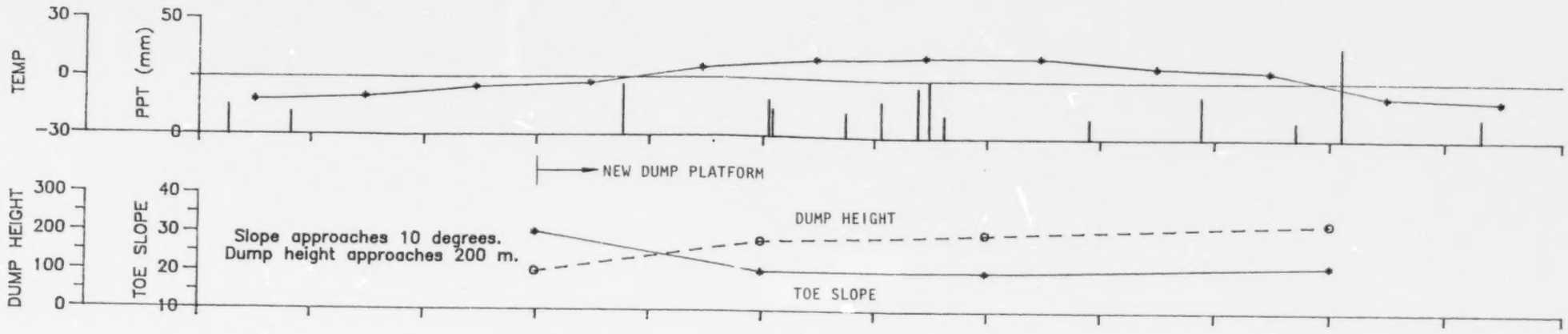
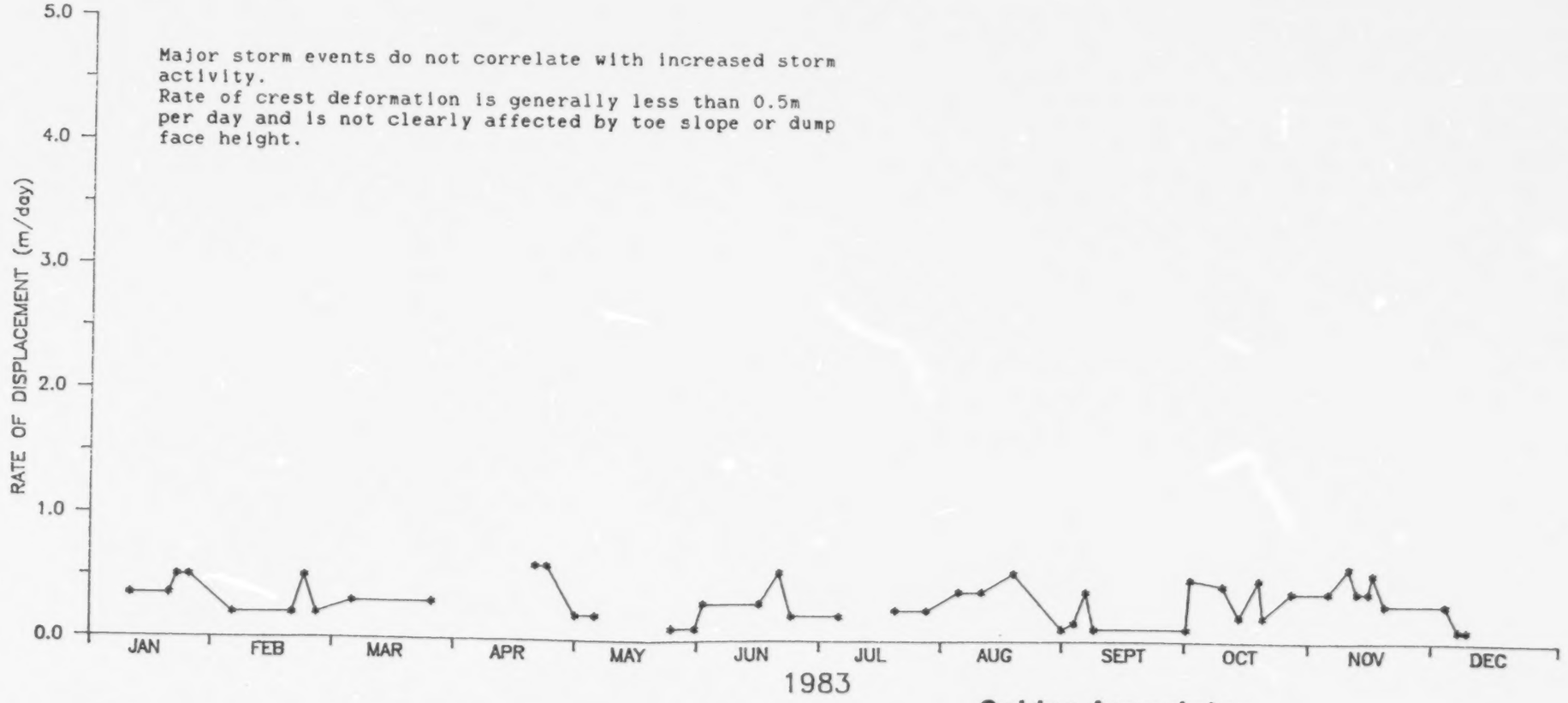
SITE B : YEAR TWO

**Figure** 5.4




Major storm events do not correlate with increased storm activity.
Rate of crest deformation is generally less than 0.5m per day and is not clearly affected by toe slope or dump face height.




PROJECT NO. ..... DRAWN ..... REVIEWED ..... DATE .....

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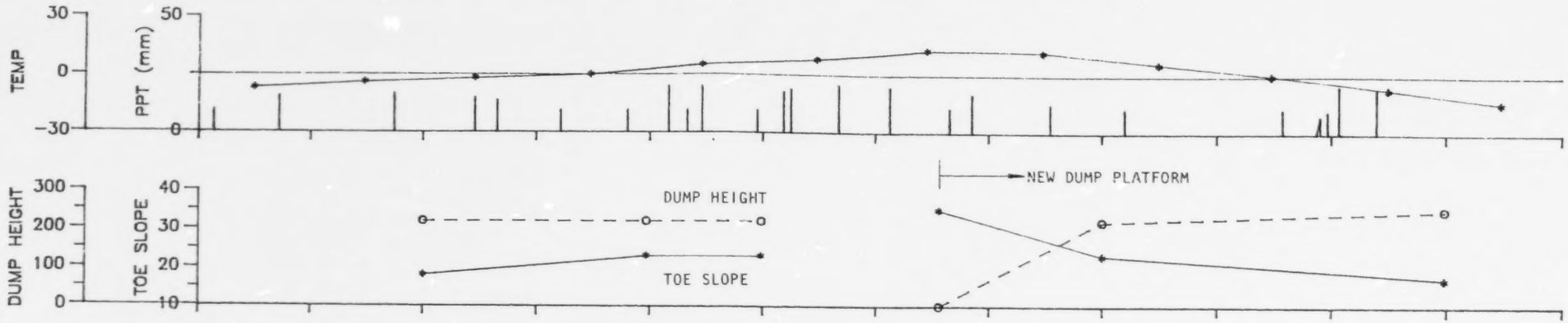
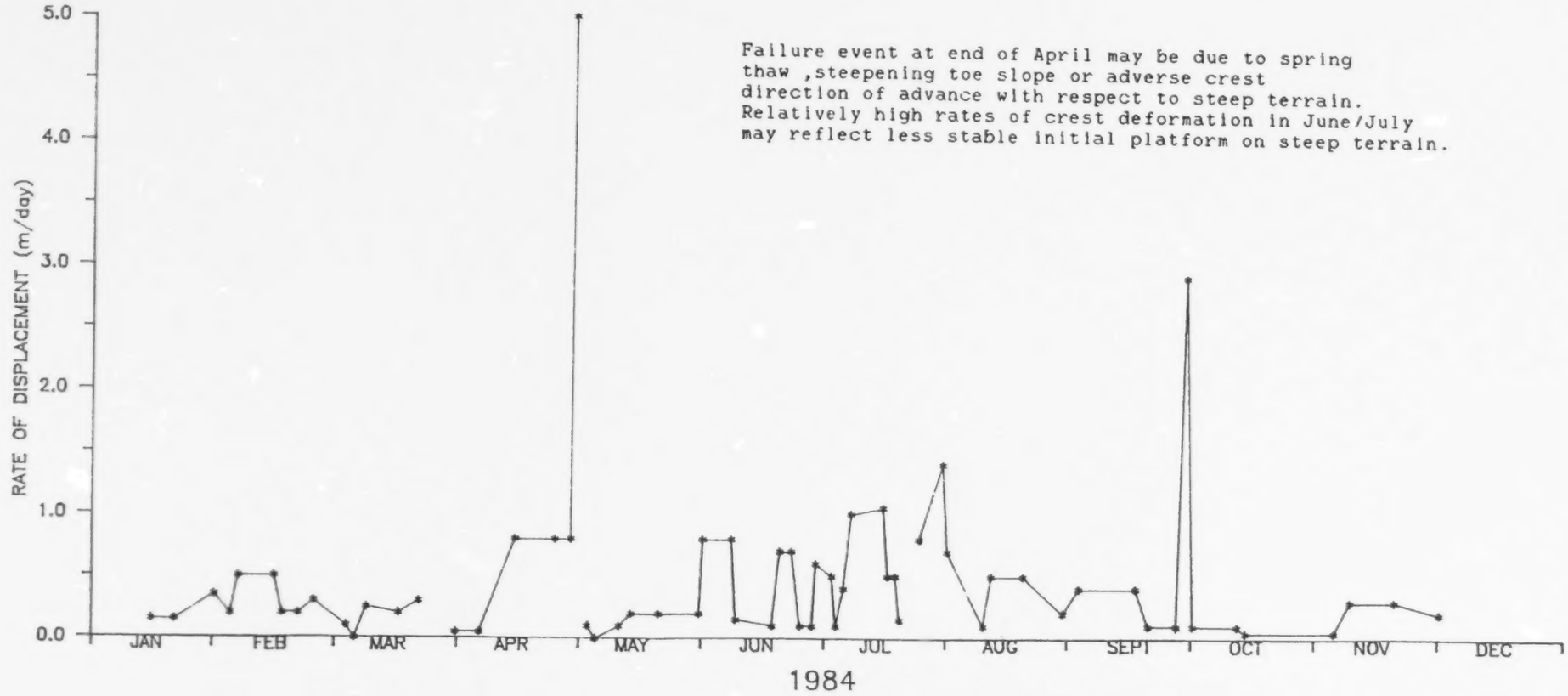
SITE B : YEAR THREE

**Figure** 5.5




Failure event at end of April may be due to spring thaw ,steepening toe slope or adverse crest direction of advance with respect to steep terrain. Relatively high rates of crest deformation in June/July may reflect less stable initial platform on steep terrain.

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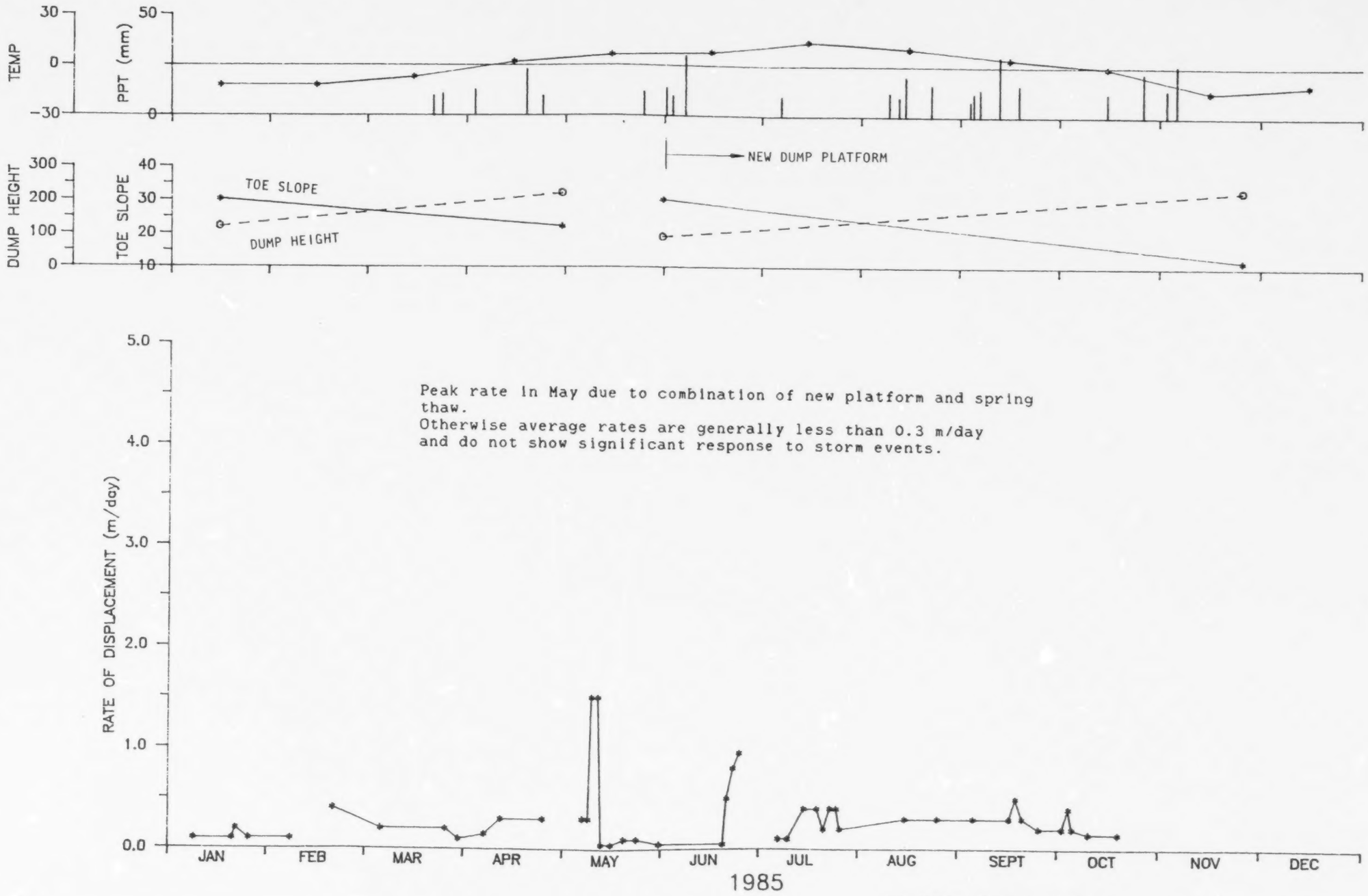
SITE B : YEAR FOUR

**Figure** 5.6




Peak rate in May due to combination of new platform and spring thaw.
Otherwise average rates are generally less than 0.3 m/day and do not show significant response to storm events.

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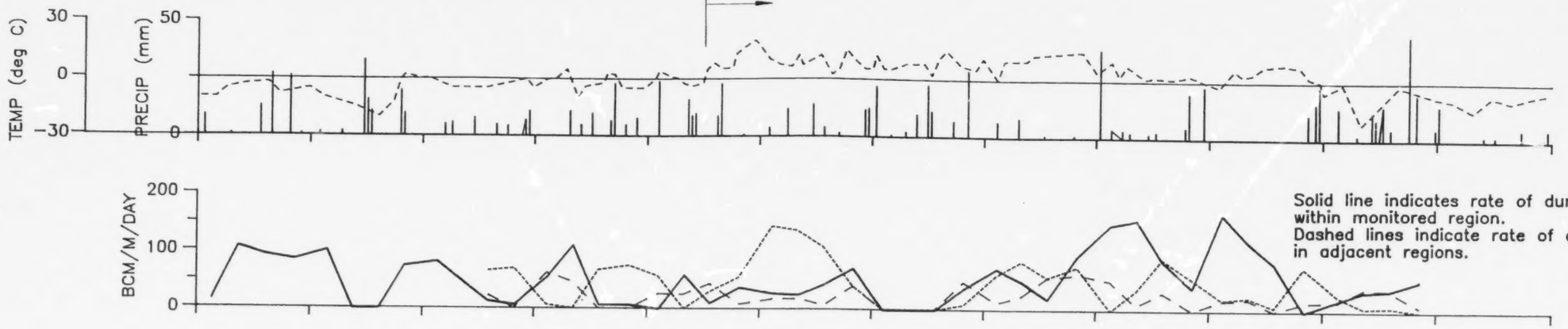
Dumping rate with respect to active crest. Solid line indicates dump rate in monitored area. Dashed lines are for adjacent areas.
Monitoring points are on crest of long sidehill fill.
Note uniform timing of peak response at each location.
High rate periods characterised by large creep movemnts below toe. No clear response to storm events.
Spring thaw appears to cause increased deformation rate.

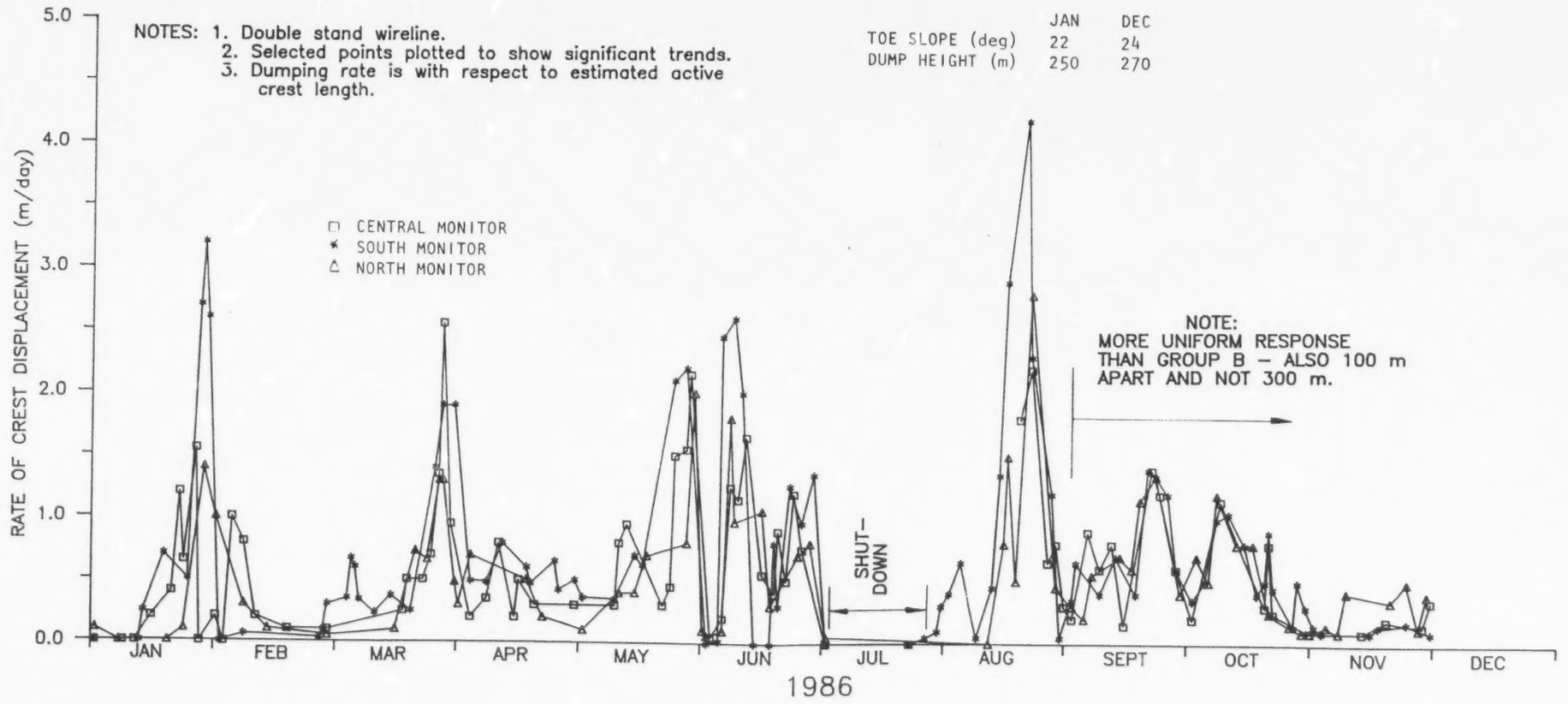
SITE C : GROUP A
1986

**Figure** 5.7




NOTES: 1. Double stand wireline.
2. Selected points plotted to show significant trends.
3. Dumping rate is with respect to estimated active crest length.

| | JAN | DEC |
|---|---|---|
| TOE SLOPE (deg) | 22 | 24 |
| DUMP HEIGHT (m) | 250 | 270 |

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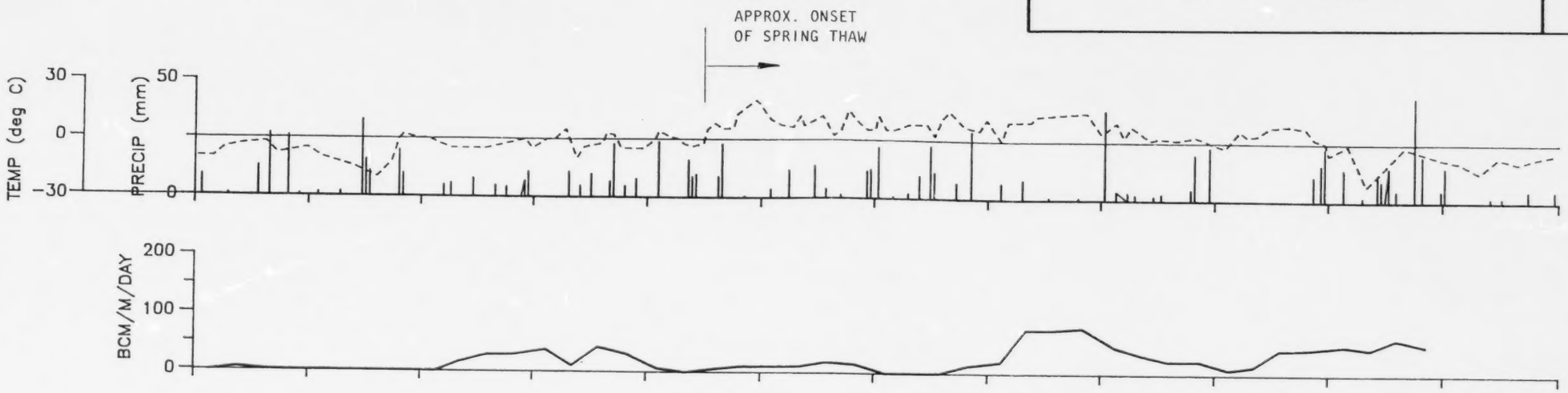
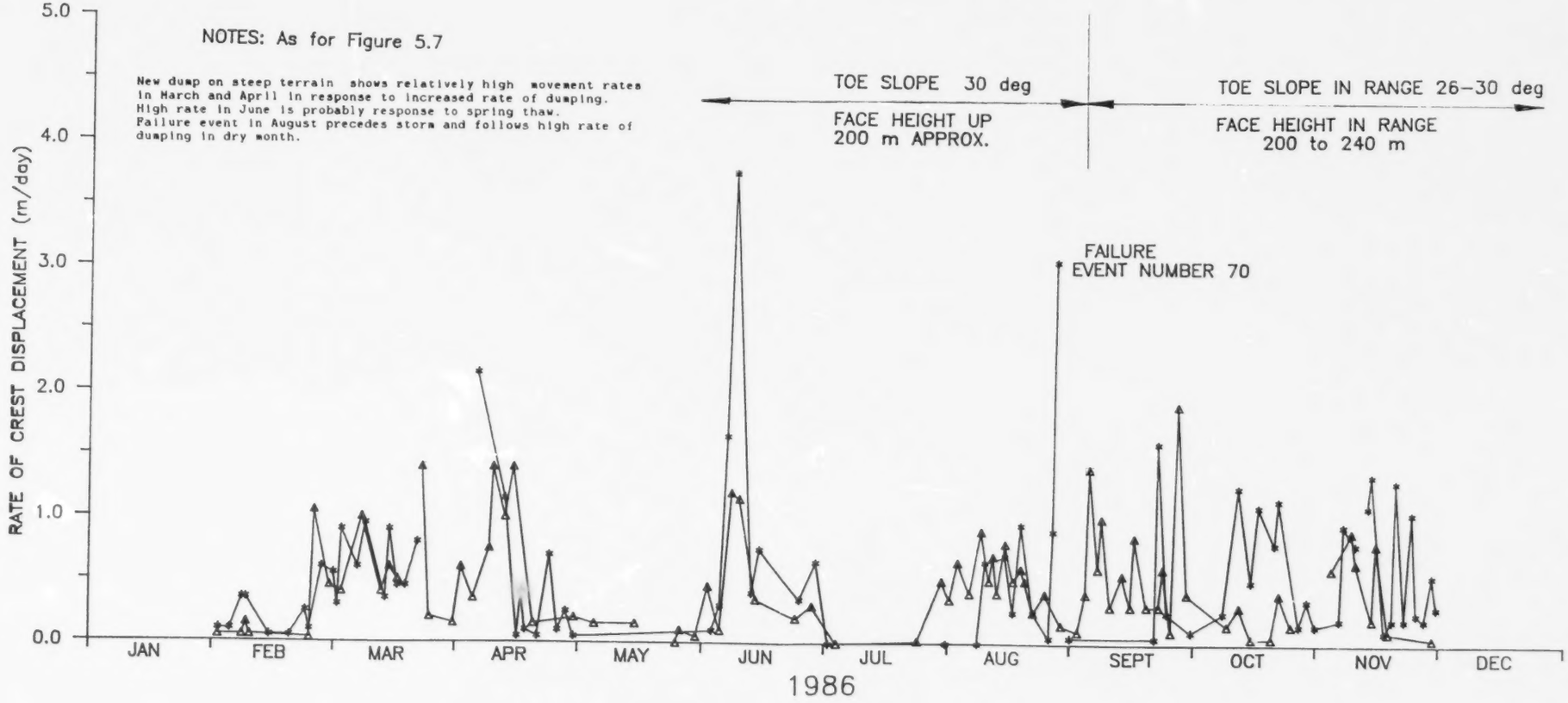
SITE C : GROUP B

**Figure** 5.8




NOTES: As for Figure 5.7

New dump on steep terrain shows relatively high movement rates in March and April in response to increased rate of dumping. High rate in June is probably response to spring thaw. Failure event in August precedes storm and follows high rate of dumping in dry month.

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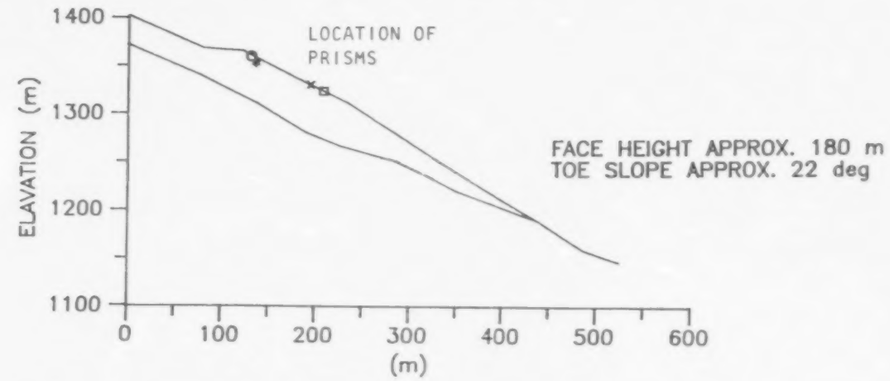
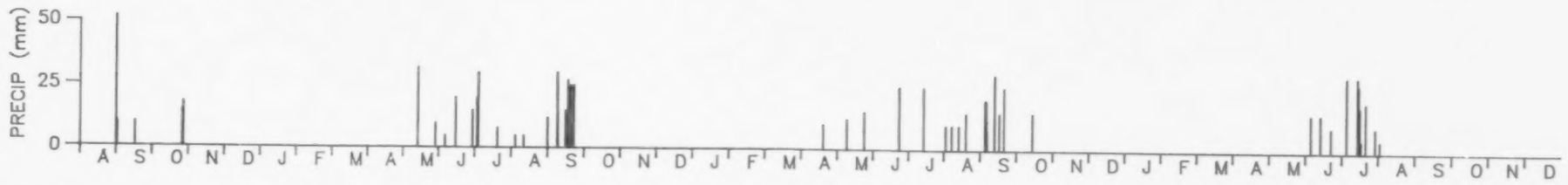
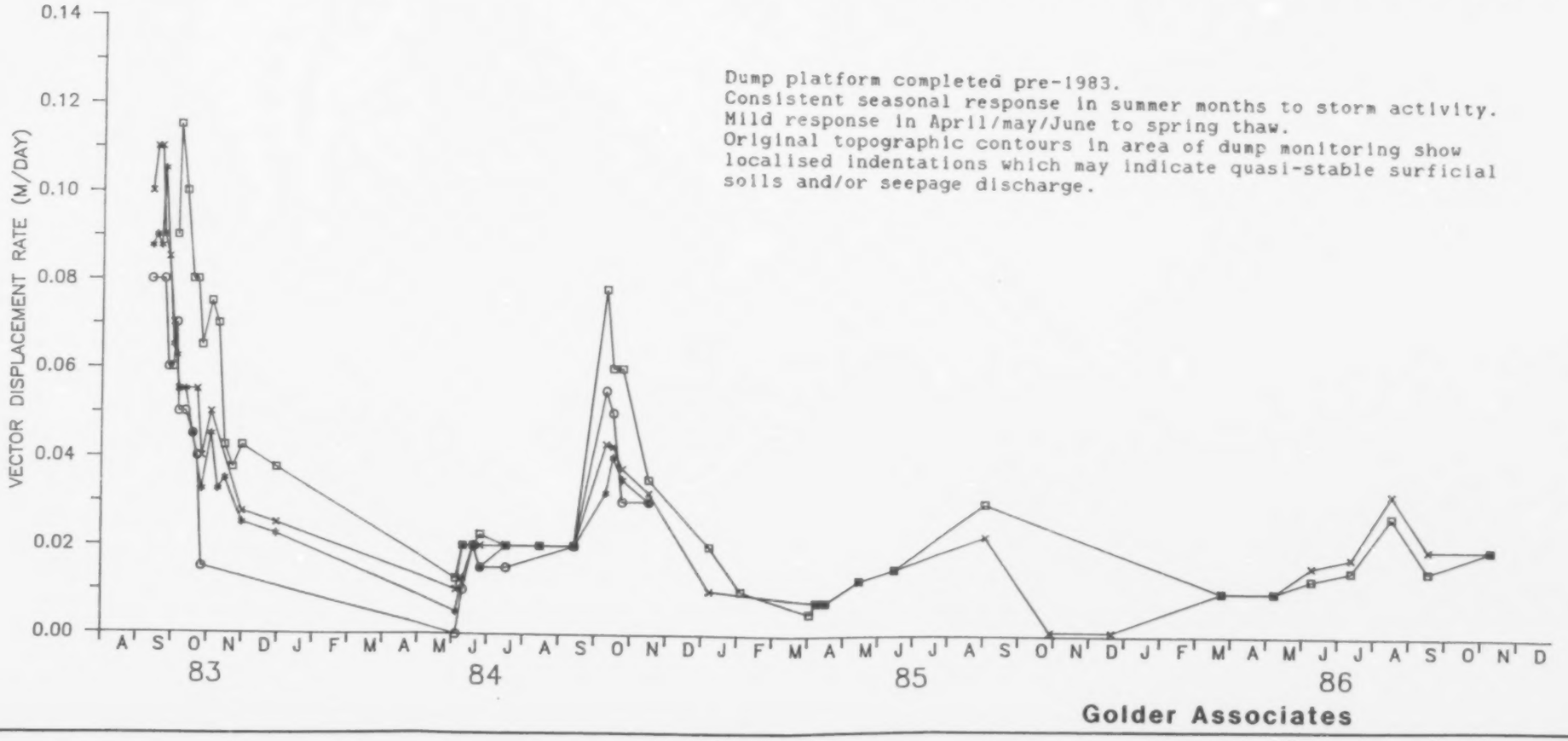
SITE H : 1983 TO 1986

**Figure** 5.9




Dump platform completed pre-1983.
Consistent seasonal response in summer months to storm activity.
Mild response in April/may/June to spring thaw.
Original topographic contours in area of dump monitoring show localised indentations which may indicate quasi-stable surficial soils and/or seepage discharge.

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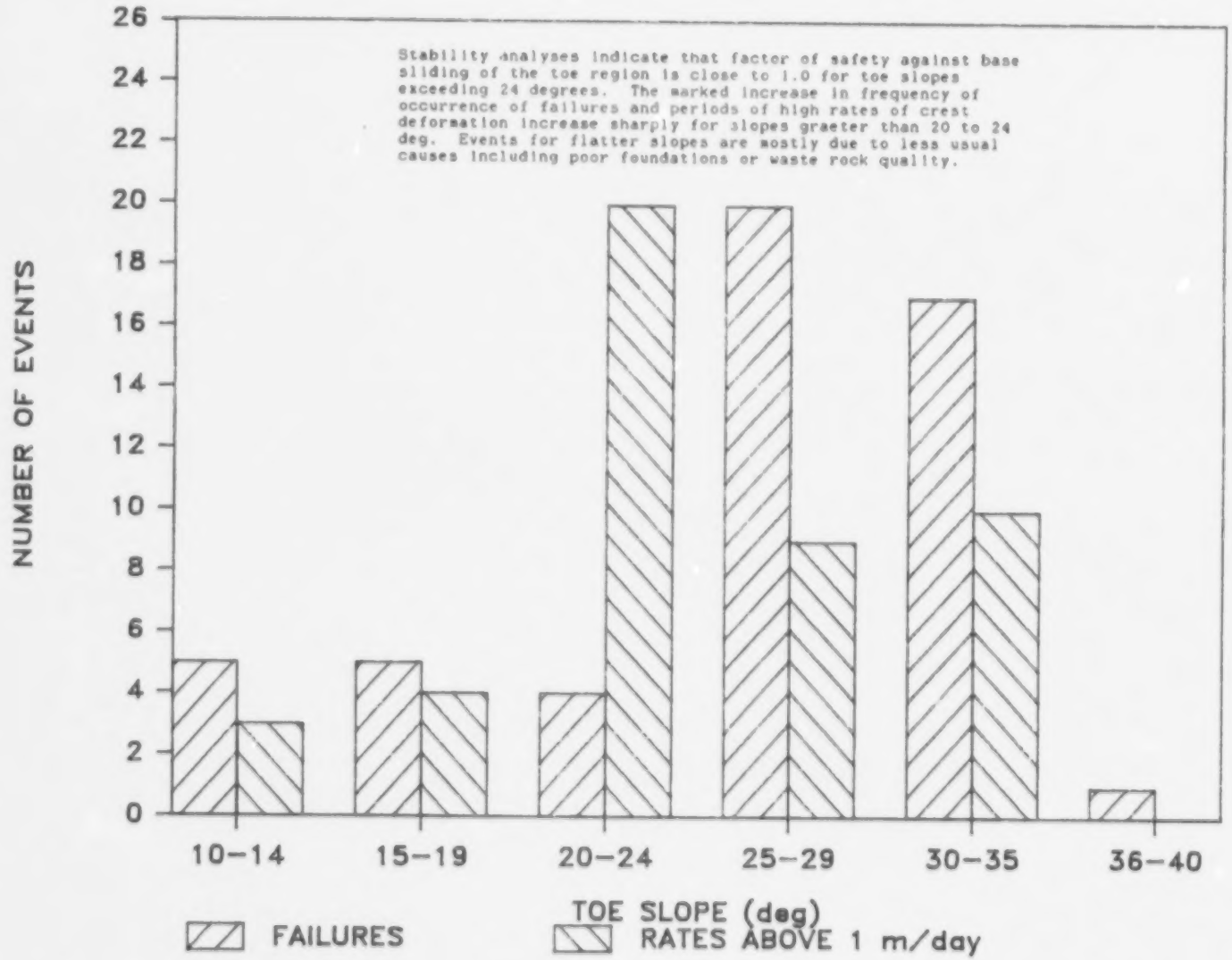
Stability analyses indicate that factor of safety against base sliding of the toe region is close to 1.0 for toe slopes exceeding 24 degrees. The marked increase in frequency of occurrence of failures and periods of high rates of crest deformation increase sharply for slopes greater than 20 to 24 deg. Events for flatter slopes are mostly due to less usual causes including poor foundations or waste rock quality.
NUMBER OF EVENTS
0
2
4
6
8
10
12
14
16
18
20
22
24
26
10–14
15–19
20–24
25–29
30–35
36–40
TOE SLOPE (deg)
FAILURES
RATES ABOVE 1 m/day

NUMBER OF EVENTS AND TOE SLOPE

Figure 5.10

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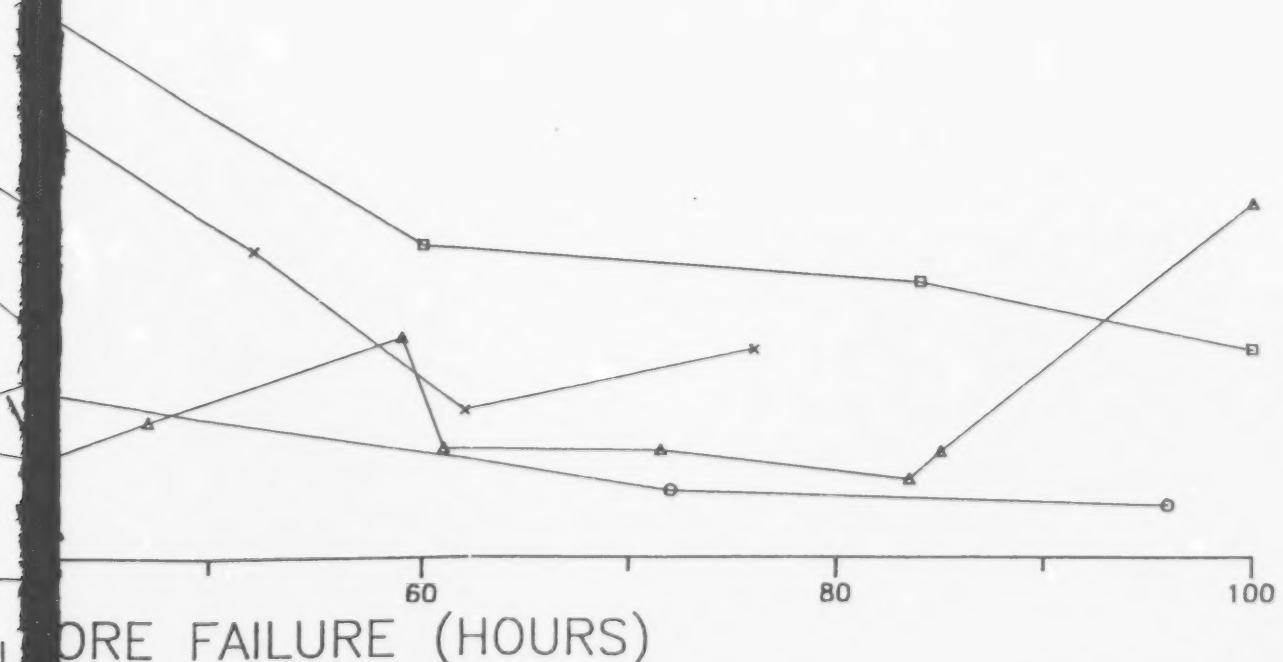
RATE OF DISPLACEMENT vs TIME PRECEDING FAILURE
TOE SLOPES > 30 deg

**Figure** 5.11

| EVENT NUMBER | TOE SLOPE (deg) | DUMP SLOPE (m) | WIRELINE TYPE | ACTIVE DUMPING IN WEEK PRIOR TO FAILURE | SUSPECTED RELATED FACTORS |
|---|---|---|---|---|---|
| □ 24 | 30 | 110 | WL2 | A | STEEPENING TOE |
| × 70 | 30 | 215 | WL2 | A | ?POOR QUALITY ROCK |
| ✱ 47 | 34 | 220 | WL1 | A | SPRING SNOWMELT |
| ○ 29 | 30+ | 150 | WL2 | A | ?CREEP |
| △ 1 | 32 | 235 | WL2 | A | ?POOR QUALITY ROCK, ?RAINFALL |

Minimum time between failure and first time that rate of crest deformation exceeds 1.0 to 1.5 m/day is in excess of 50 hours.

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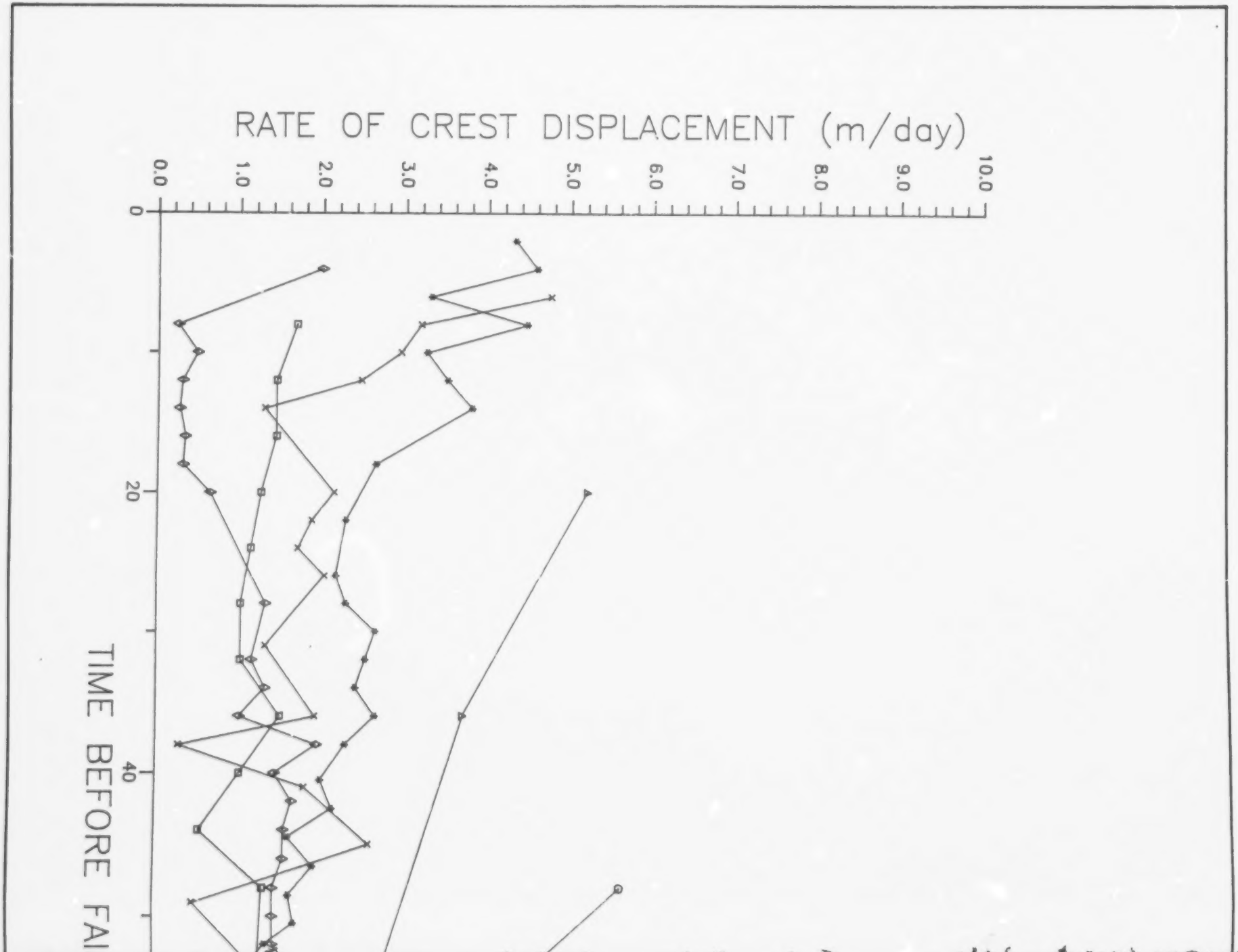
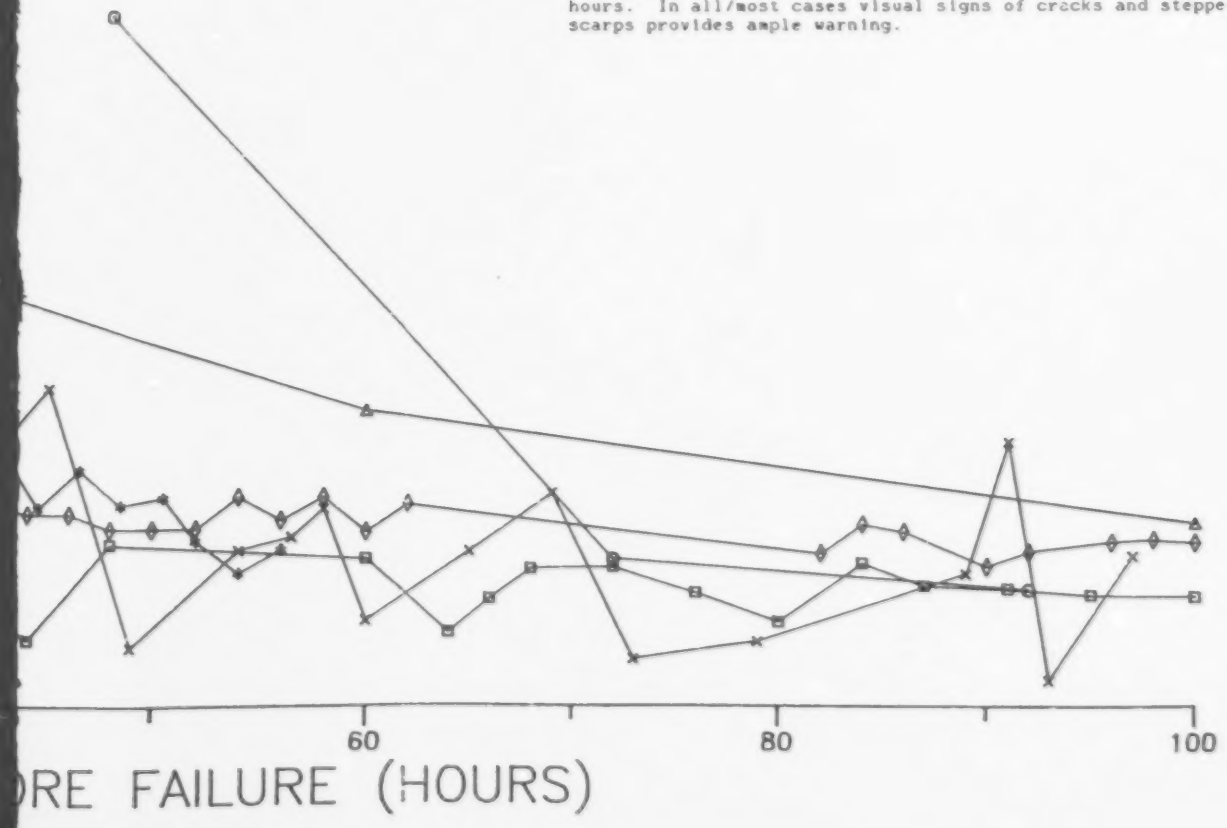


RATE OF DISPLACEMENT vs TIME PRECEDING FAILURE
TOE SLOPES 24 -30 deg

**Figure** 5.12

| EVENT NUMBER | TOE SLOPE (deg) | DUMP SLOPE (m) | WIRELINE TYPE | ACTIVE DUMPING IN WEEK PRIOR TO FAILURE | SUSPECTED RELATED FACTORS |
|---|---|---|---|---|---|
| ○ | 75 | 27 | 140 | WL2 | A | POOR ROCK QUALITY, FOUNDATION |
| ✳ | 69 | 28 | 190 | WL2 | A | SPRING SNOWMELT |
| × | 3 | 26 | 225 | WL2 | A | RAINFALL |
| □ | 5 | 26 | 210 | WL2 | A | SPRING SNOWMELT |
| ◇ | 11 | 28 | 300 | WL2 | A | POOR ROCK QUALITY, RAINFALL |
| △ | 88 | 29 | 130 | WL2 | A | ? |

As shown on fugure 5.16, average time to failure past crest deformation rate of 1.0 m/day is 80 hours with a minimum of 40 hours. In all/most cases visual signs of cracks and stepped scarps provides ample warning.

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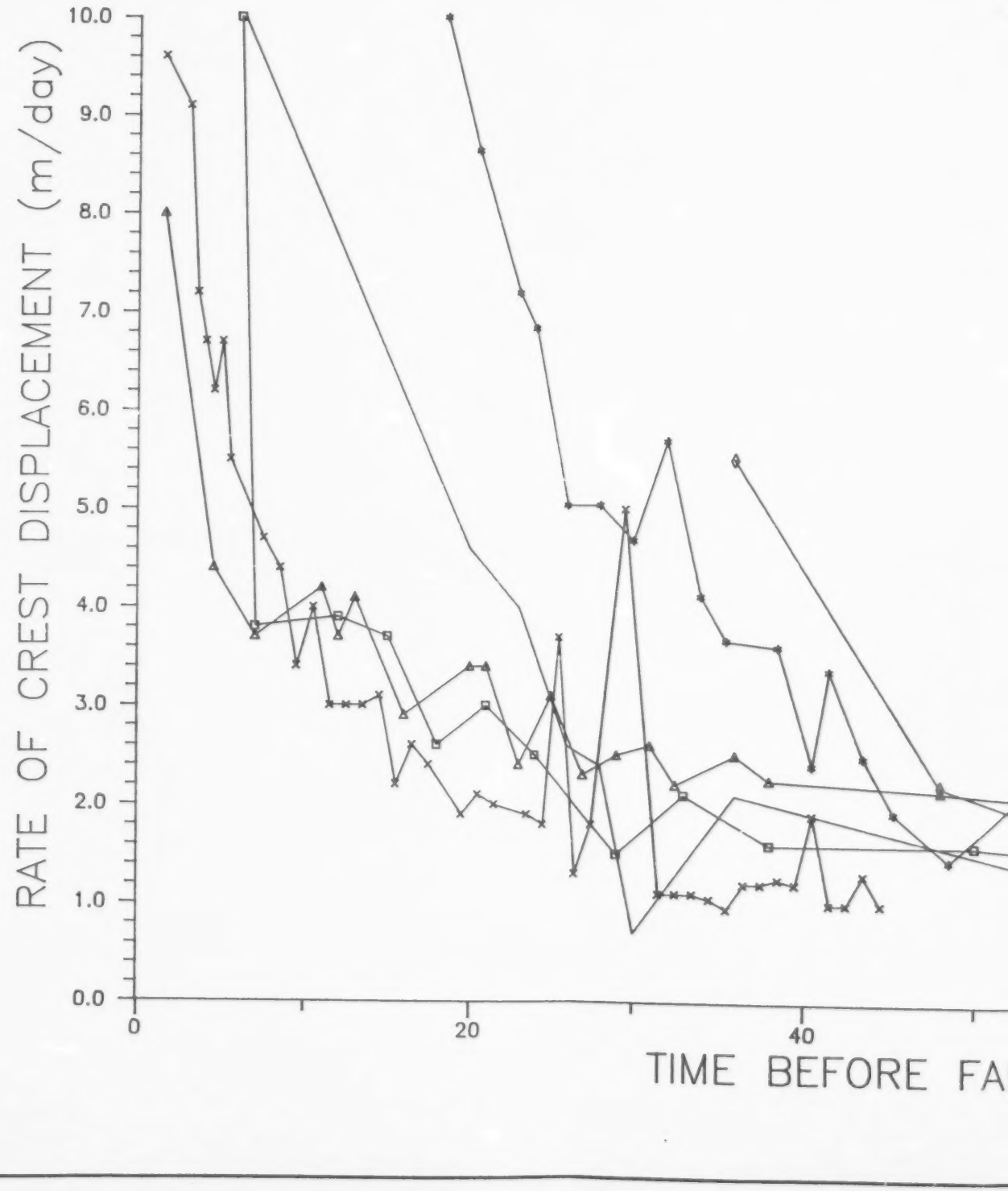


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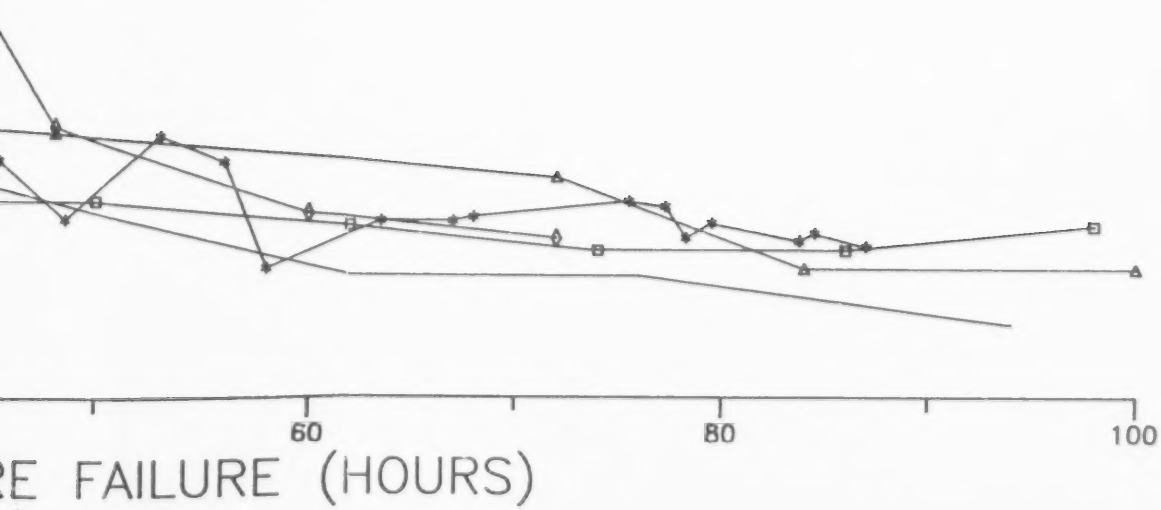
RATE OF DISPLACEMENT vs TIME PRECEDING FAILURE
TOE SLOPES 11 - 20 deg

**Figure** 5.13

| | EVENT NUMBER | TOE SLOPE (deg) | DUMP SLOPE (m) | WIRELINE TYPE | ACTIVE DUMPING IN WEEK PRIOR TO FAILURE | SUSPECTED RELATED FACTORS |
|---|---|---|---|---|---|---|
| * | 9 | 11 | 240 | WL2 | A | ? |
| × | 12 | 20 | 180 | WL2 | I | ?SNOW MELT,INACTIVE |
| □ | 48 | 14 | 200 | WL1 | A | ?POOR ROCK QUALITY |
| △ | 49 | 16 | 180 | WL1 | A | TOE STEEPENS, RAINFALL, ROCK QUALITY |
| ◇ | 113 | 21 | 250 | WL1 | A | ?RAINFALL, ?FOUNDATION |
| ○ | 117 | 12 | 220 | WL2 | A | SPRING SNOWMELT, FNDN, TOE STEEPENS |

Relatively high acceleration compared to trends for steeper slopes shown on figures 5.11 and 5.12. This may reflect more rapid loss of strength due to fine gradation and/or pore pressure generation. Average time to failure past rate of 1.0 m/day is 80 hours.

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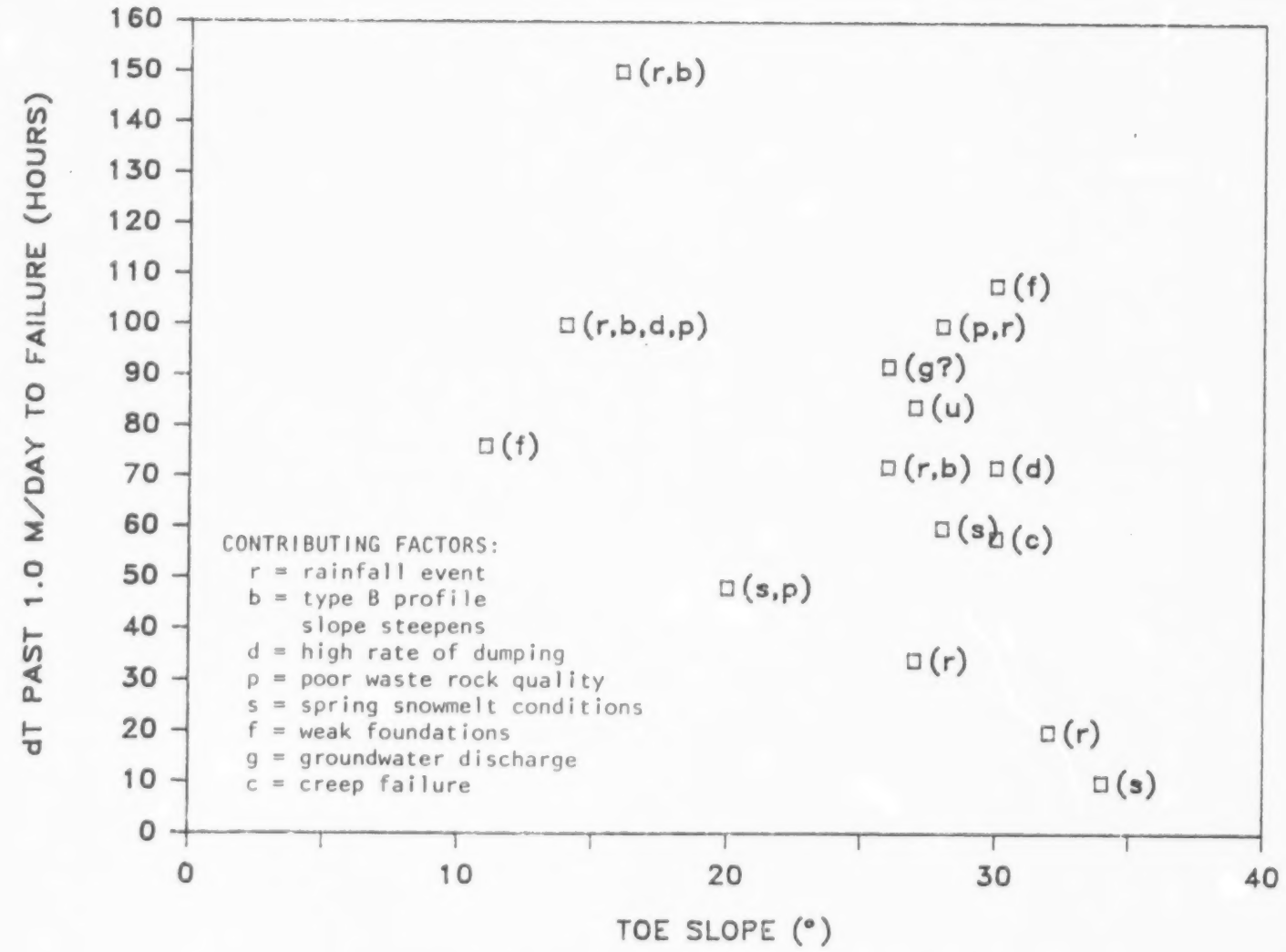


A lower bound of extreme values indicates a trend of decreasing time to failure with increasing toe slope. Apart from toe slope a common factors identified as being potential contributing factors is rainfall. Excluding two points for slopes greater than 30 deg, minimum time past 1.0 m/day is 30 hours.

TIME PAST RATE OF 1.0 M/DAY TO FAILURE vs TOE SLOPE

Figure 5.14

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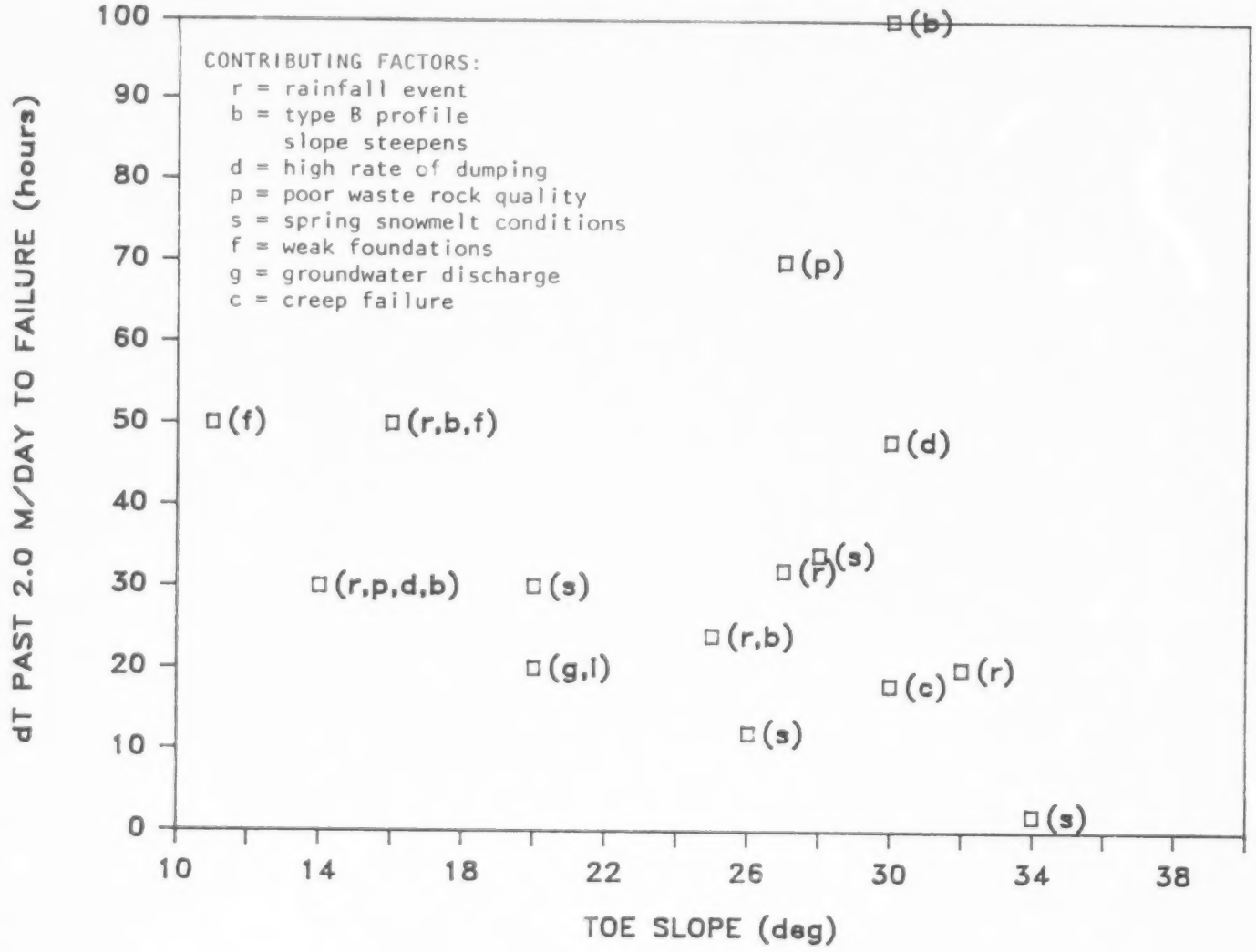


Similar to figure 5.14 but shows that minimum time from rate of 2.0 m/day is almost always greater than 10 hours.

TIME PAST RATE OF 2.0 M/DAY TO FAILURE vs TOE SLOPE

Figure 5.15

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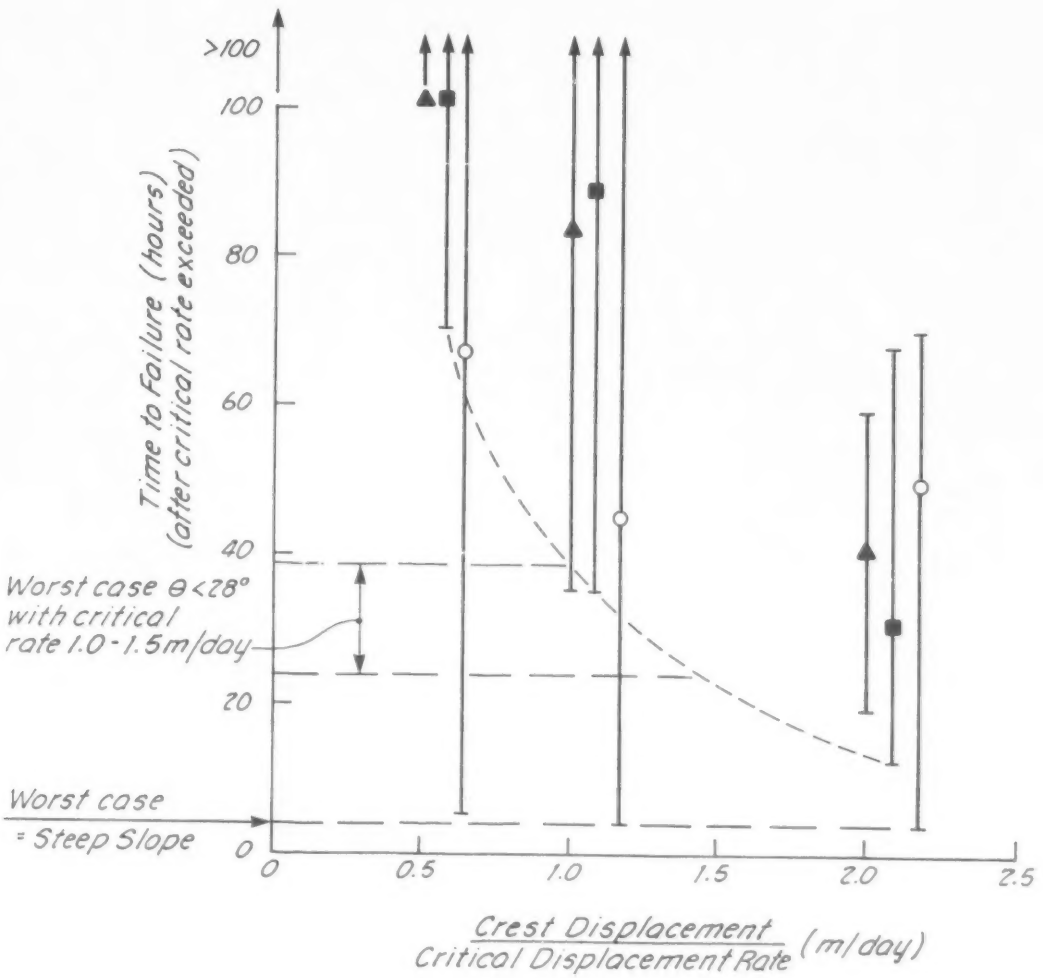
TIME TO FAILURE PAST SELECTED RATE OF CREST DISPLACEMENT

Figure **5.16**

Summarises plots shown on figures 5.11, 5.12 and 5.12.. Average values show a minimum time to failure past 2.0 m/day of approximately 30 hours. Worst case (event no 47) is unusual and dotted lower bound shown is considered to be more reasonable indication of lowest likely values of time to failure.




LEGEND:

Range

Average Values:

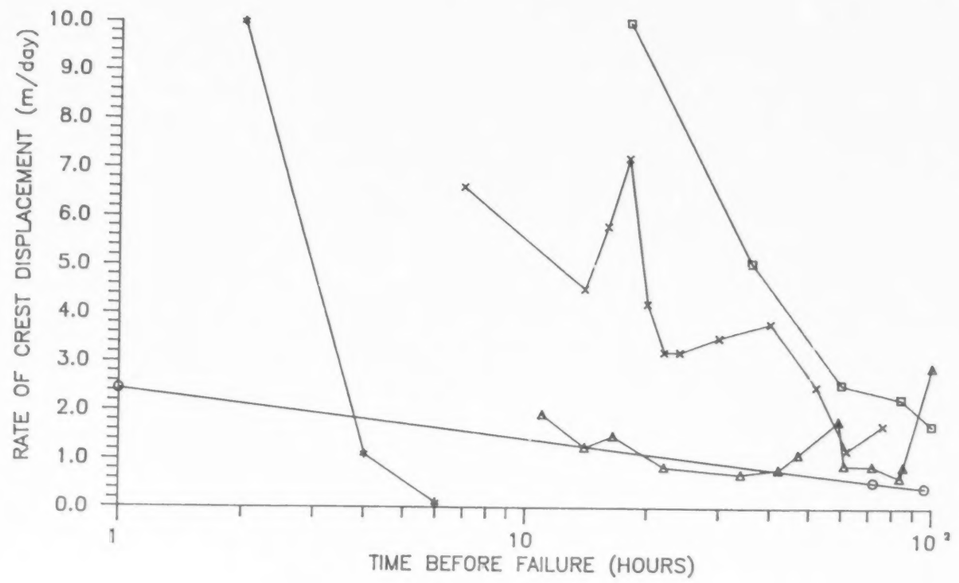
- ▲ Toe Slope 14-20°
- ■ Toe Slope 24-28°
- ○ Toe Slope >29°

} Average of 4-7 plots

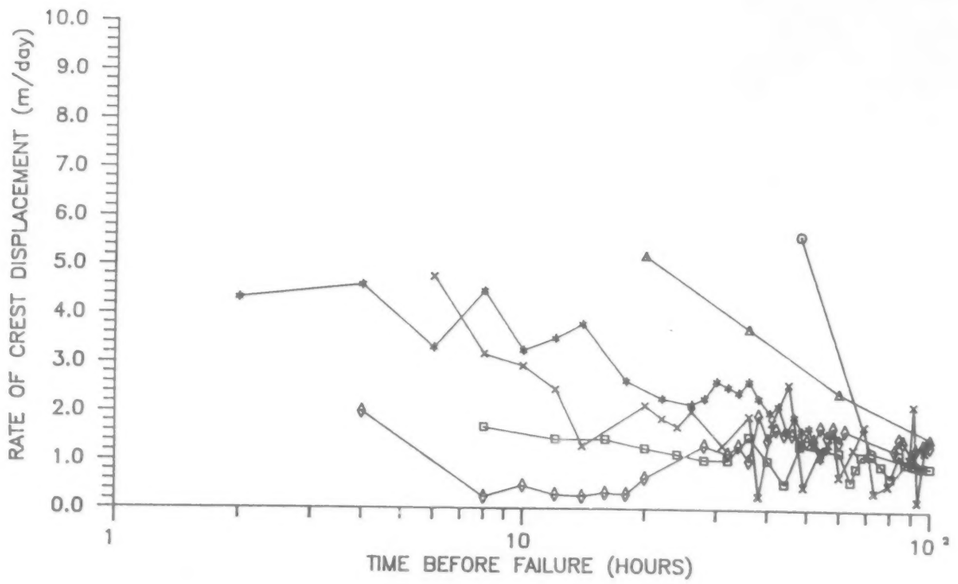
PROJECT NO. 862-1231.E DRAWN BAO REVIEWED DATE May '87

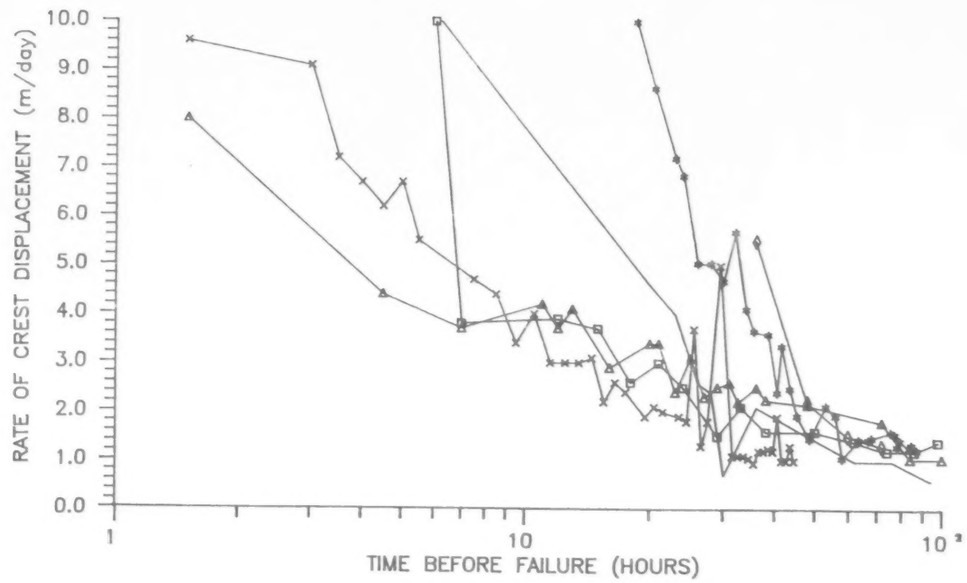
CREST RATE OF DISPLACEMENT vs
LOGARITHM OF TIME BEFORE FAILURE

**Figure** 5.17




Plots performed in order to distinguish varying rates of acceleration. Plots for steeper and flatter slope ranges show higher rates of acceleration than intermediate range of toe slopes.

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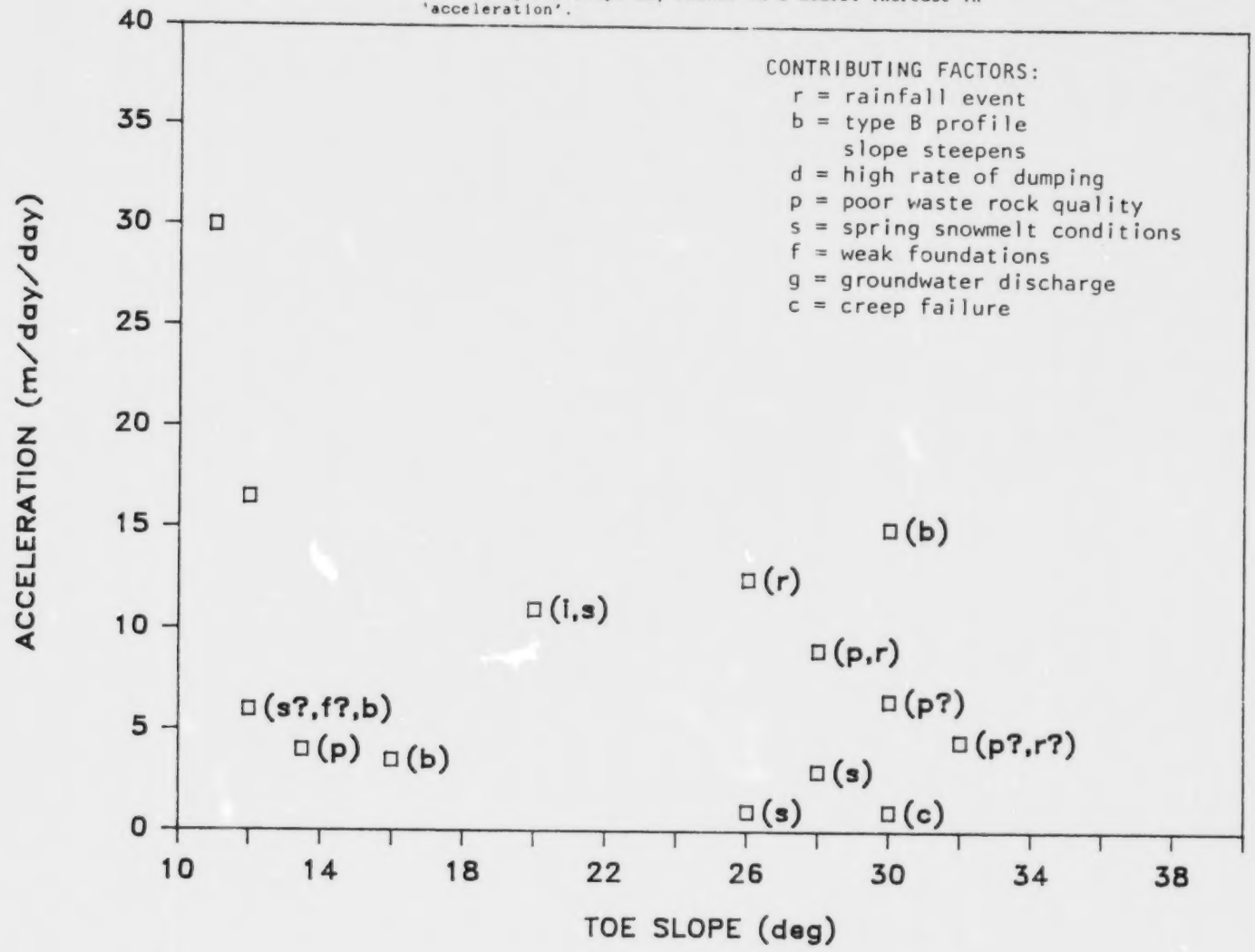
The term 'acceleration' is used on the ordinate to denote the slope of the plot of rate of crest deformation against logarithm of time before failure shown on figurer 5.17. Highest values of acceleration arer for flattest slopes. Such failures are associated with seepage and fine-grained waste rock. Otherwise increasing toe slope may result in a modest increase in 'acceleration'.




SLOPE OF CREST RATE / LOG TIME PLOT vs TOE SLOPE

Figure 5.18

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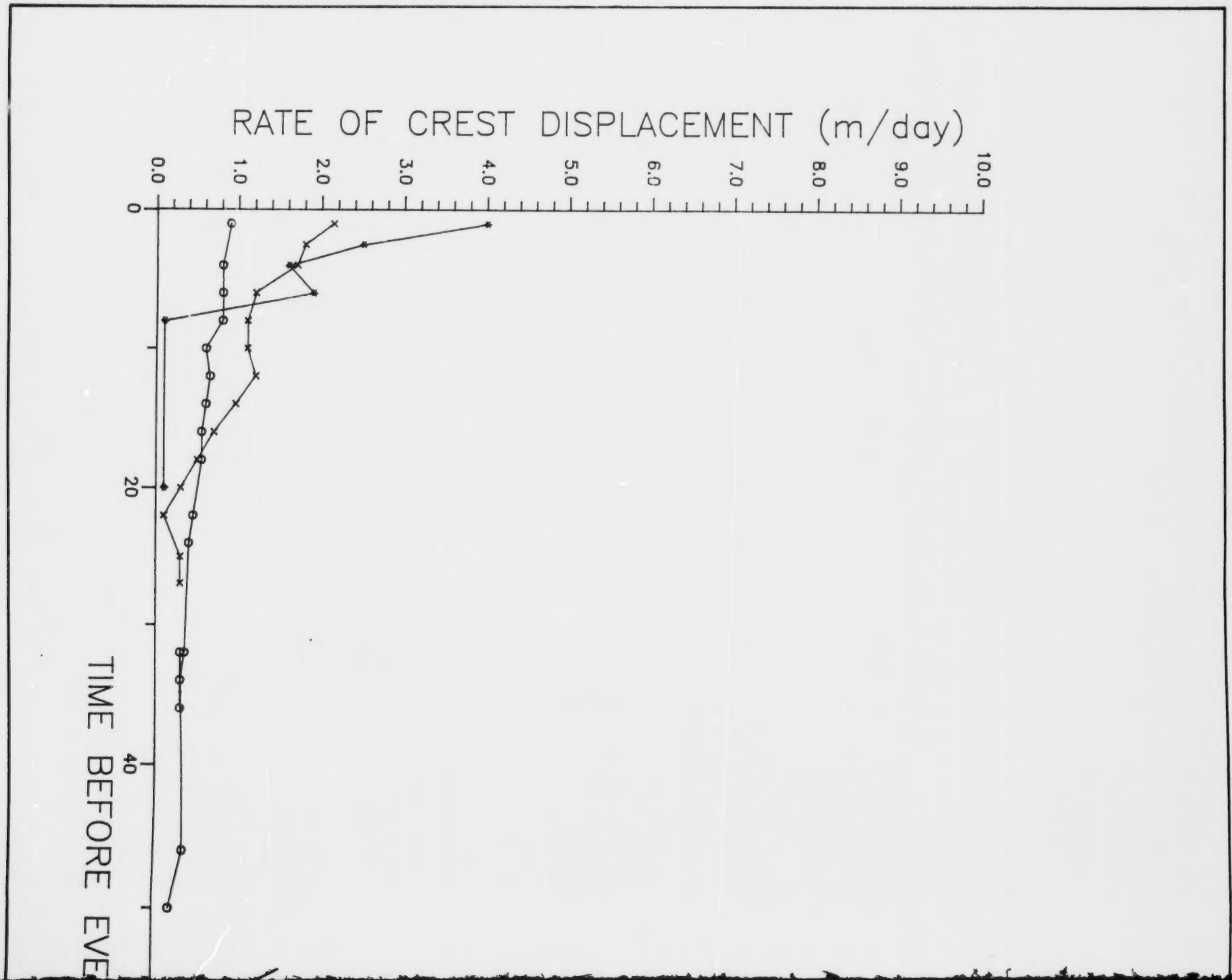
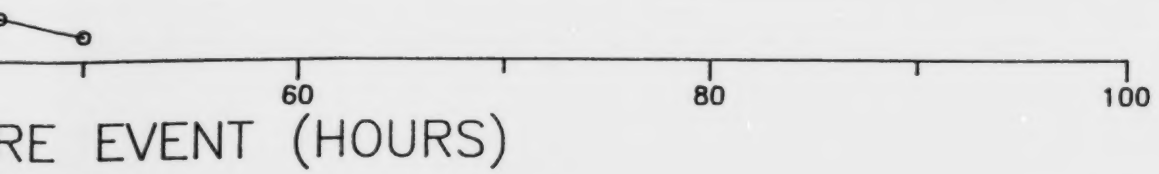


CREST DISPLACEMENT RATE vs TIME TO PEAK
NON-FAILURE EVENTS ON STEEP TOE SLOPES

**Figure** 5.19

| | EVENT NUMBER | TOE SLOPE (deg) | DUMP SLOPE (m) | WIRELINE TYPE | SUSPECTED RELATED FACTORS |
|---|---|---|---|---|---|
| * | 109 | 30 | 265 | WL2 | DUMPING RATE |
| × | 107 | 30 | 250 | WL2 | DUMPING RATE, ?RAINFALL |
| O | 63 | 30 | 190 | WL2 | DUMPING RATE |

Peak rates of crest displacement of 2.0 to 4.0 m/day can develop within a 20 hour period. Dumping was halted as rate passed 0.5 m/day signifying satisfactory reaction by operations personnel.

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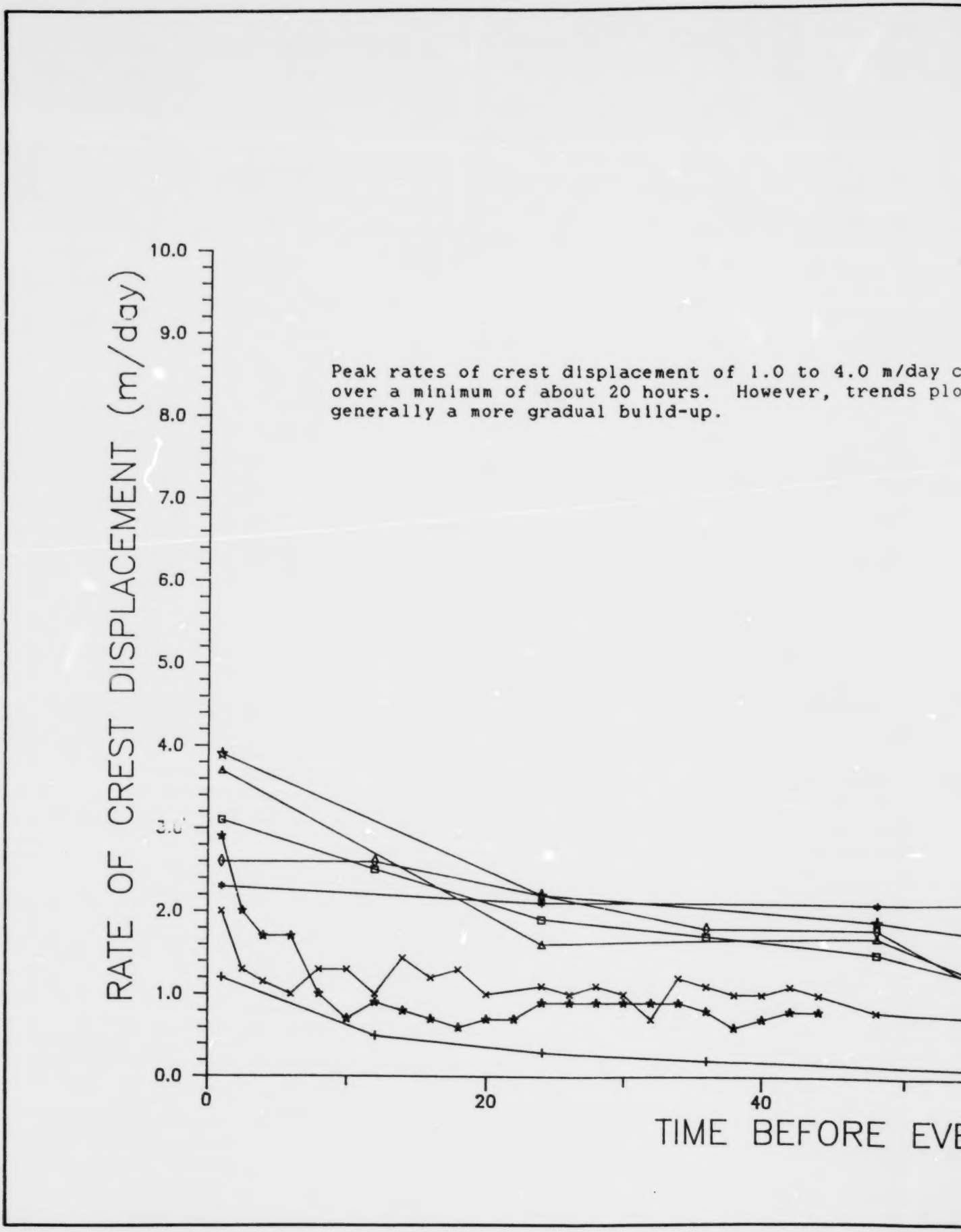
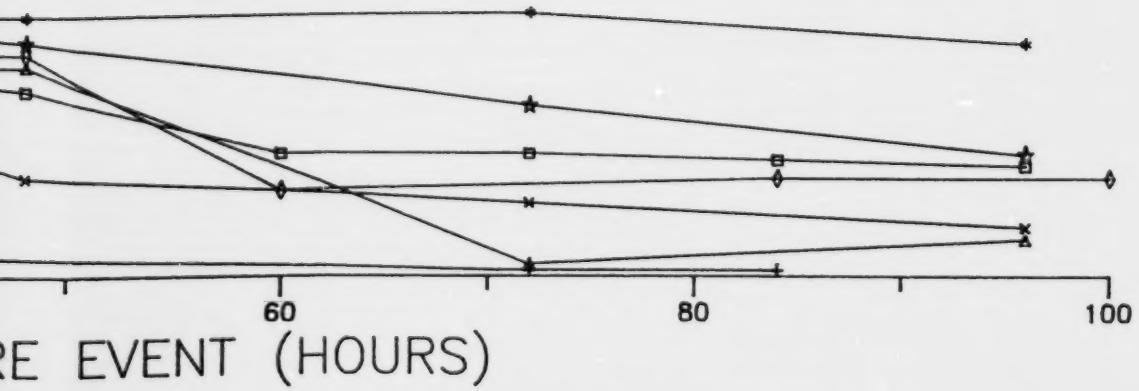

Peak rates of crest displacement of 1.0 to 4.0 m/day c
over a minimum of about 20 hours. However, trends plo
generally a more gradual build-up.
RATE OF CREST DISPLACEMENT (m/day)
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
20
40
TIME BEFORE EVE
PROJECT NO.
DRAWN
REVIEWED
DATE

CREST DISPLACEMENT RATE vs TIME TO PEAK
NON-FAILURE EVENTS ON MODERATE TOE SLOPES

**Figure** 5.20

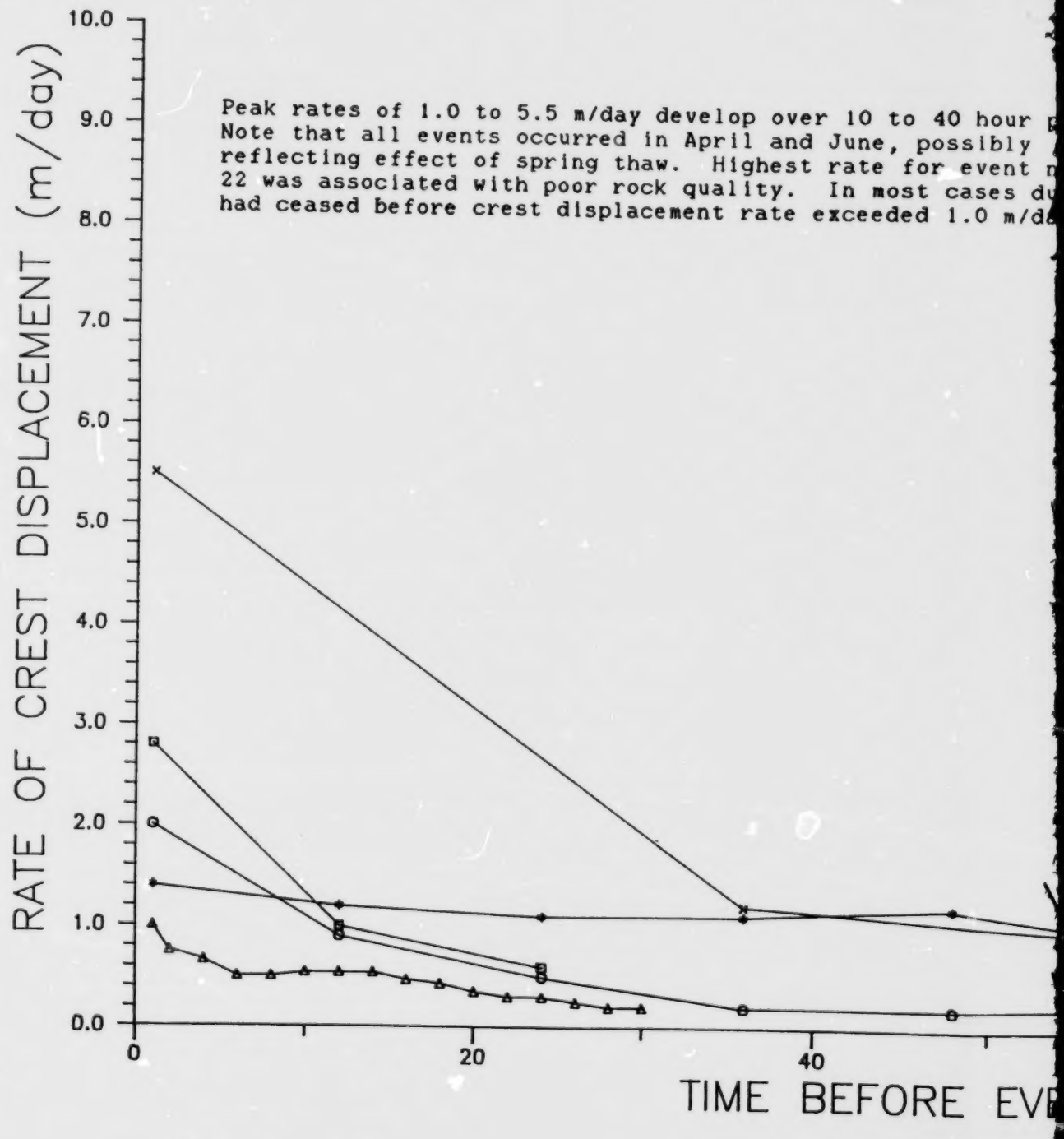
0 m/day can develop
rends plotted show

| | EVENT NUMBER | TOE SLOPE (deg) | DUMP SLOPE (m) | WIRELINE TYPE | SUSPECTED RELATED FACTORS |
|---|---|---|---|---|---|
| ✳ | 92 | 23 | 220 | WL2 | SPRING SNOWMELT, RAINFALL |
| ○ | 91 | 23 | 220 | WL2 | SPRING SNOWMELT |
| □ | 56 | 24 | 220 | WL1 | RAINFALL |
| ◇ | 50 | 24 | 225 | WL1 | ?DUMPING RATE |
| + | 57 | 22 | 190 | WL1 | DUMPING RATE |
| ☆ | 94 | 23 | 220 | WL2 | DUMPING RATE |
| ★ | 120 | 25 | 220 | WL2 | POOR ROCK QUALITY, ?DUMPING RATE |
| △ | 105 | 20 | 230 | WL2 | DUMPING RATE |
| × | 100 | 23 | 260 | WL2 | DUMPING RATE |




RE EVENT (HOURS)

**Golder Associates**




Peak rates of 1.0 to 5.5 m/day develop over 10 to 40 hour p
Note that all events occurred in April and June, possibly
reflecting effect of spring thaw. Highest rate for event n
22 was associated with poor rock quality. In most cases du
had ceased before crest displacement rate exceeded 1.0 m/da

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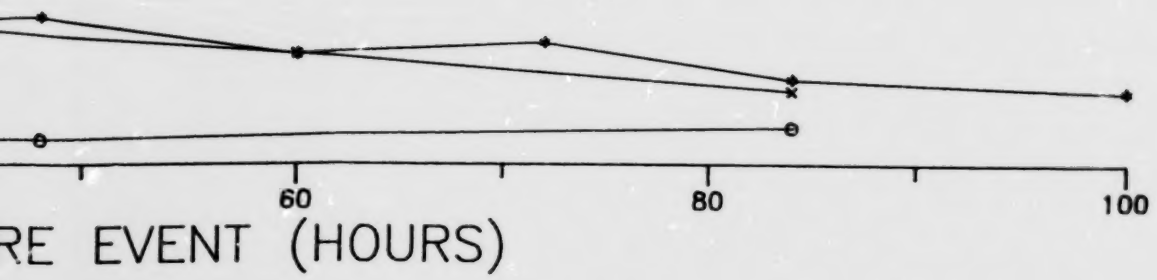
CREST DISPLACEMENT RATE vs TIME TO PEAK
NON-FAILURE EVENTS ON 10 - 20 DEG SLOPES

**Figure** 5.21

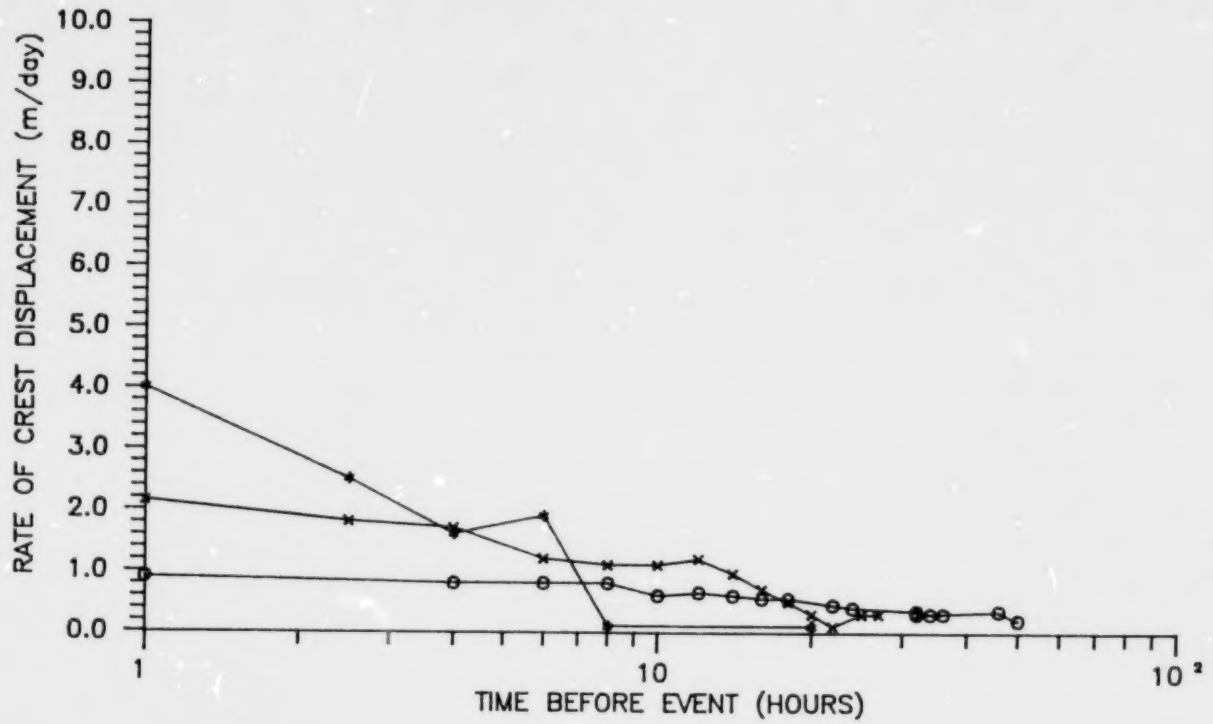
40 hour period.
possibly
or event number
t cases dumping
d 1.0 m/day.

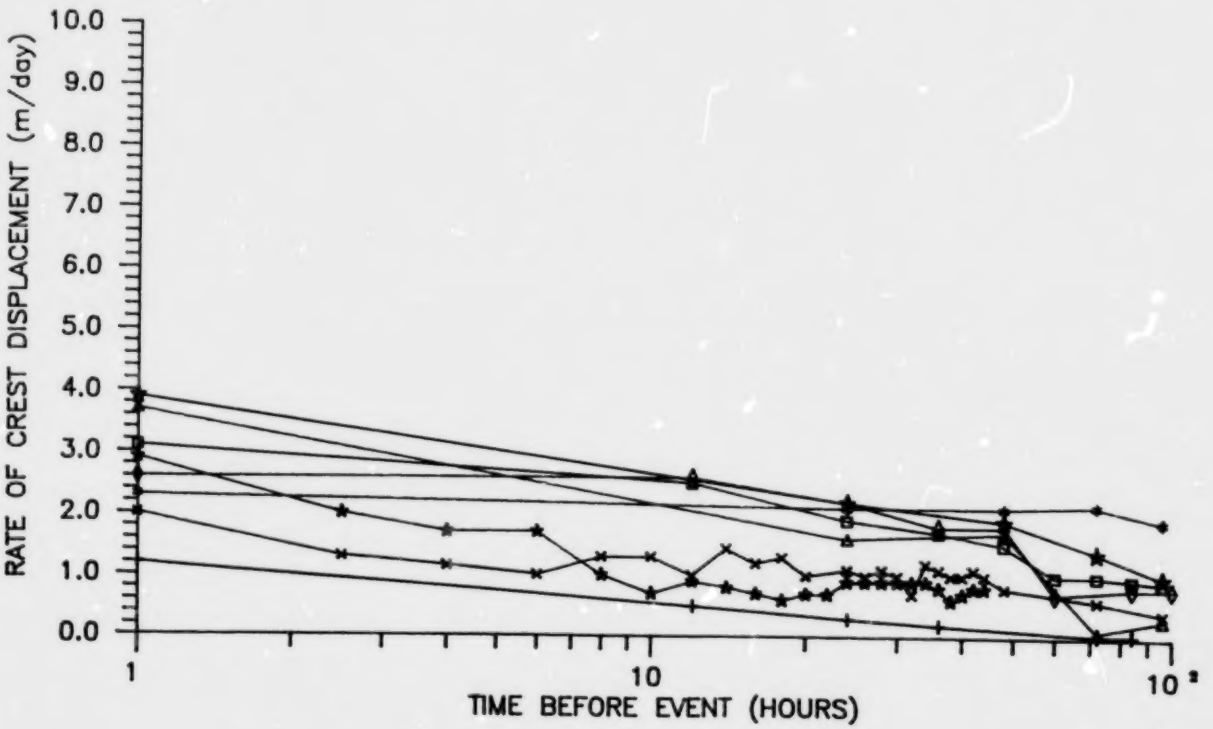
| | EVENT NUMBER | TOE SLOPE (deg) | DUMP SLOPE (m) | WIRELINE TYPE | SUSPECTED RELATED FACTORS |
|---|---|---|---|---|---|
| * | 52 | 14 | 190 | WL1 | SPRING SNOWMELT, DUMPING RATE |
| □ | 51 | 14 | 190 | WL1 | RAINFALL |
| ○ | 58 | 23 | 210 | WL1 | ?RAINFALL, DUMPING RATE |
| △ | 44 | 20 | 180 | WL2 | DUMPING RATE? |
| × | 22 | 15-20 | 150-200 | WL2 | POOR ROCK QUALITY, SPRING SNOWMELT |




RE EVENT (HOURS)

Golder Associates

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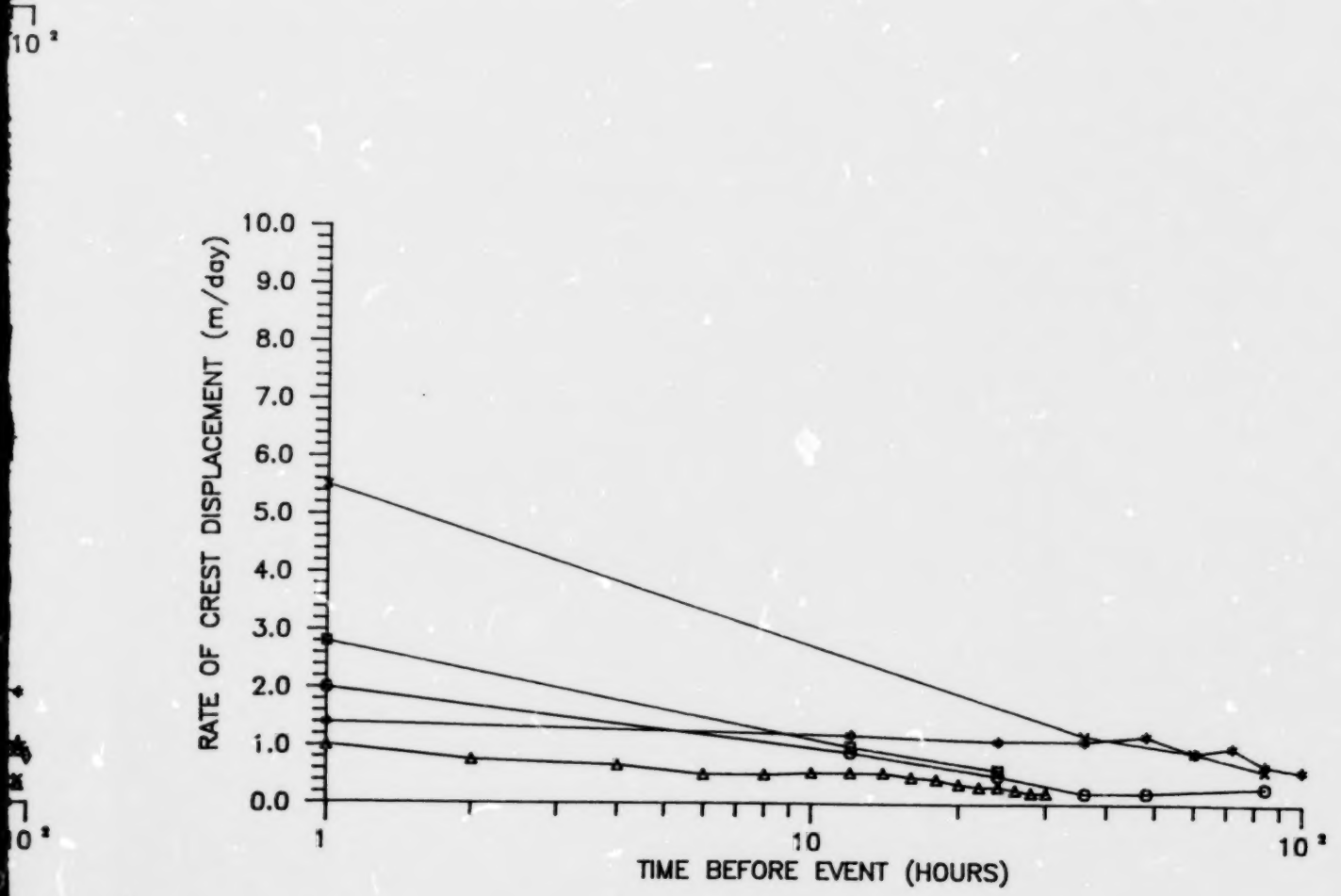
CREST DISPLACEMENT RATE vs LOGARITHM OF TIME TO PEAK RATE - NON FAILURE EVENTS

**Figure** 5.22

Moderate slopes show a relatively constant rate of increase in crest displacement rate. For steeper and flatter slopes a marked steepening of plot can occur 10 to 40 hours prior to peak.

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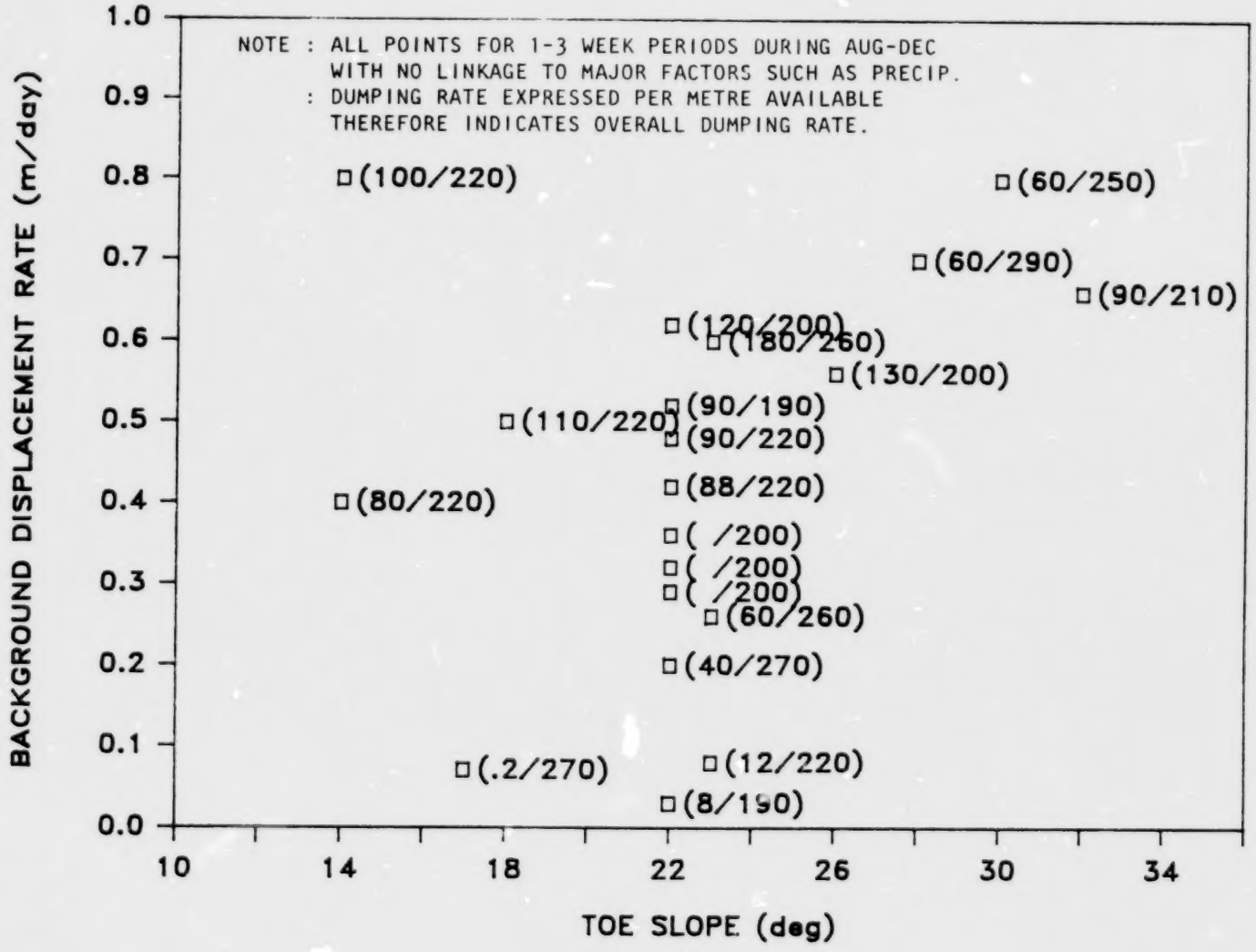
PROJECT NO. ..... DRAWN ..... REVIEWED ..... DATE .....

Background rate is taken over 1-2 week period. Rate of dumping appears to have more effect on displacement rate than dump height or toe slope. Upper bound of data points plotted may show a trend of increasing displacement with higher toe slope.



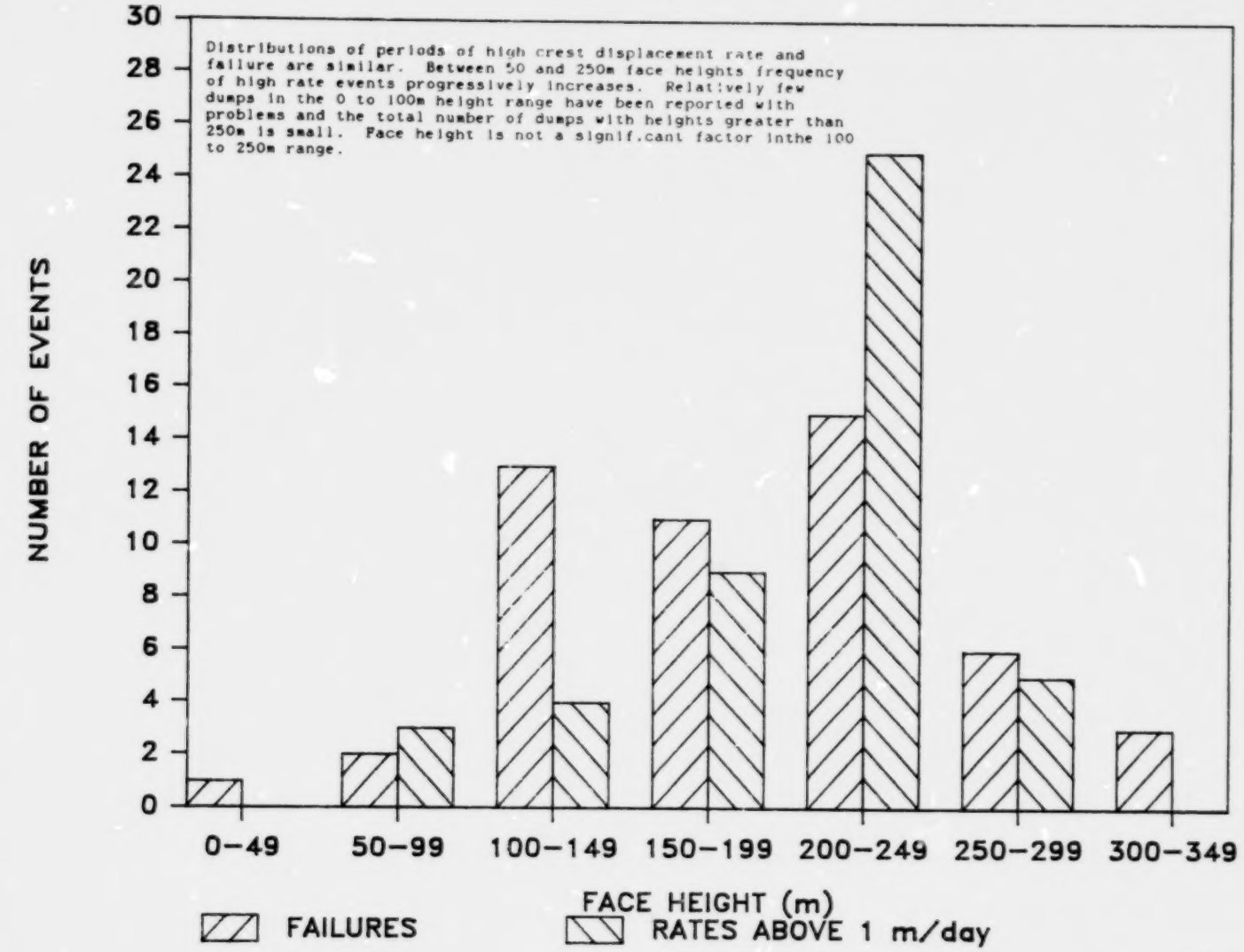

(X,Y)
X = RATE OF DUMPING (bcm/m/day)
Y = DUMP HEIGHT (m)

BACKGROUND (NORMAL) CREST DISPLACEMENT RATE vs TOE SLOPE

Figure 5.23

Golder Associates

PROJECT NO. ..... DRAWN ..... REVIEWED ..... DATE .....




NUMBER OF SIGNIFICANT EVENTS vs DUMP FACE HEIGHT

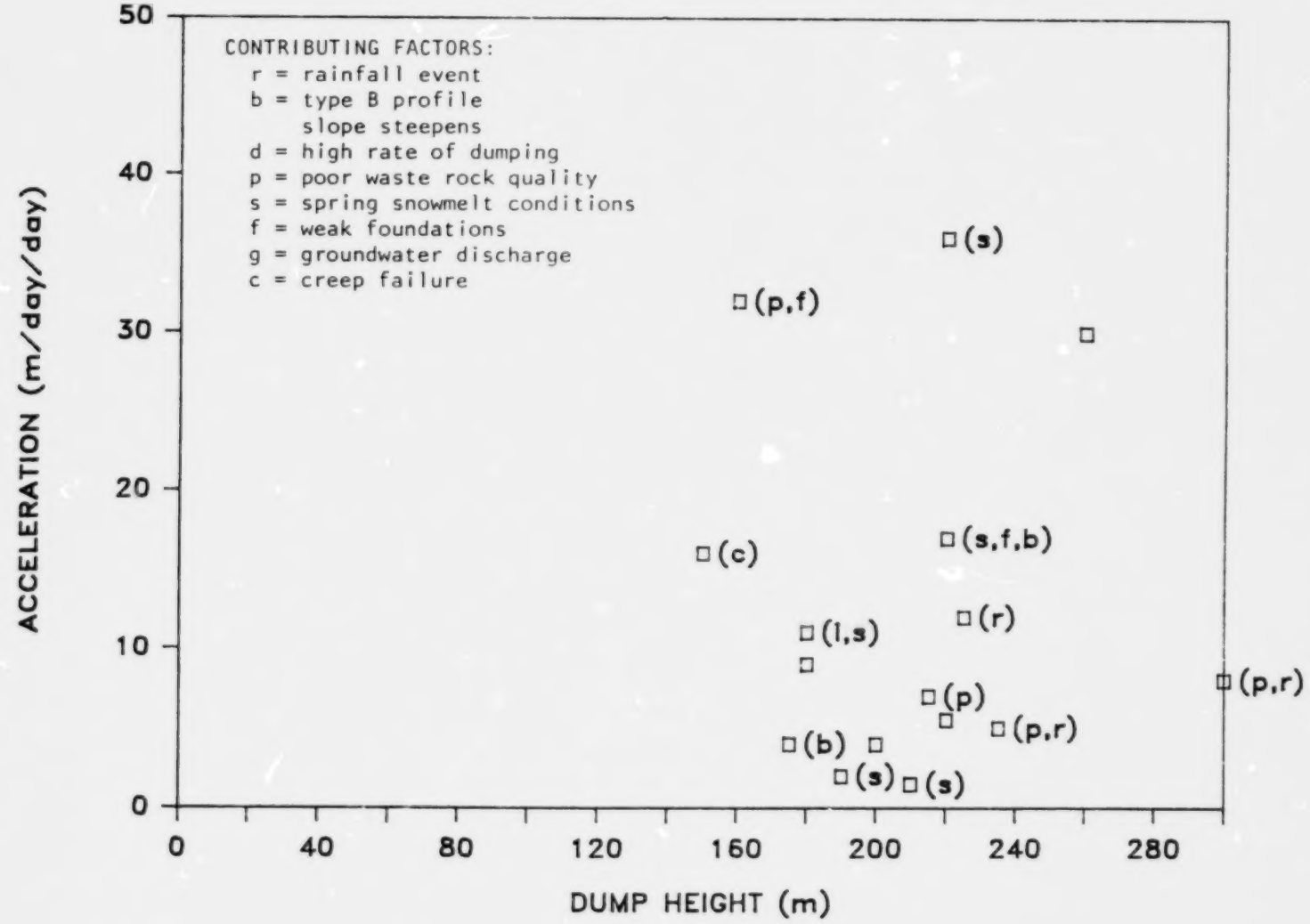
Figure 5.24

Golder Associates

PROJECT NO. DRAWN REVIEWED DATE

Golder Associates

Reliable data for dump heights less than 100m is not available.
Value of 'acceleration' is not clearly linked to specific causative factors.

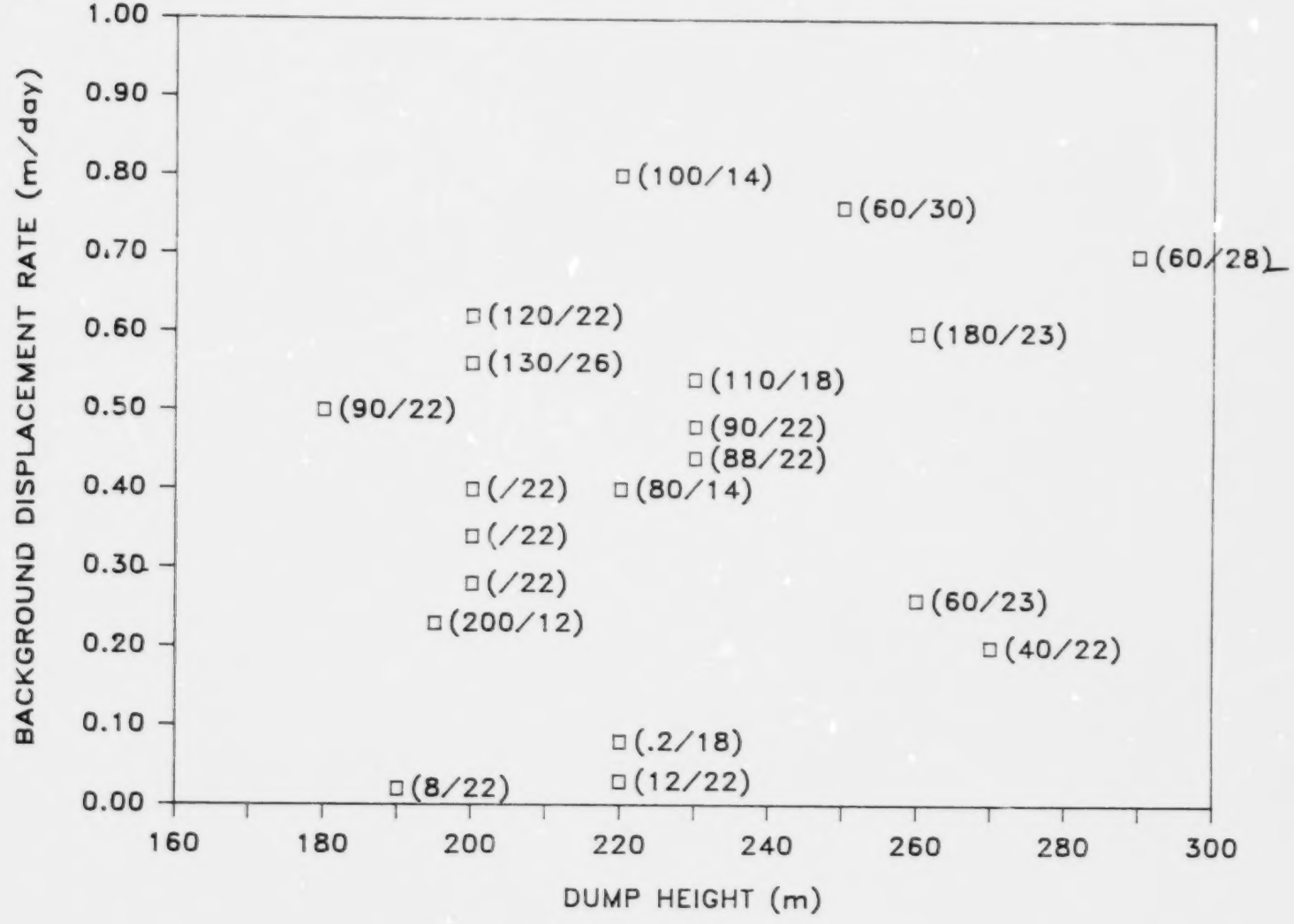



ACCELERATION FOR NON-FAILURE EVENTS vs DUMP HEIGHT

Figure 5.25

PROJECT NO. ..... DRAWN ..... REVIEWED ..... DATE .....

Higher rate of dumping appears to have most significant effect on background or normal rate of crest displacement. Lowest and highest toe slopes correspond to highest background rate.



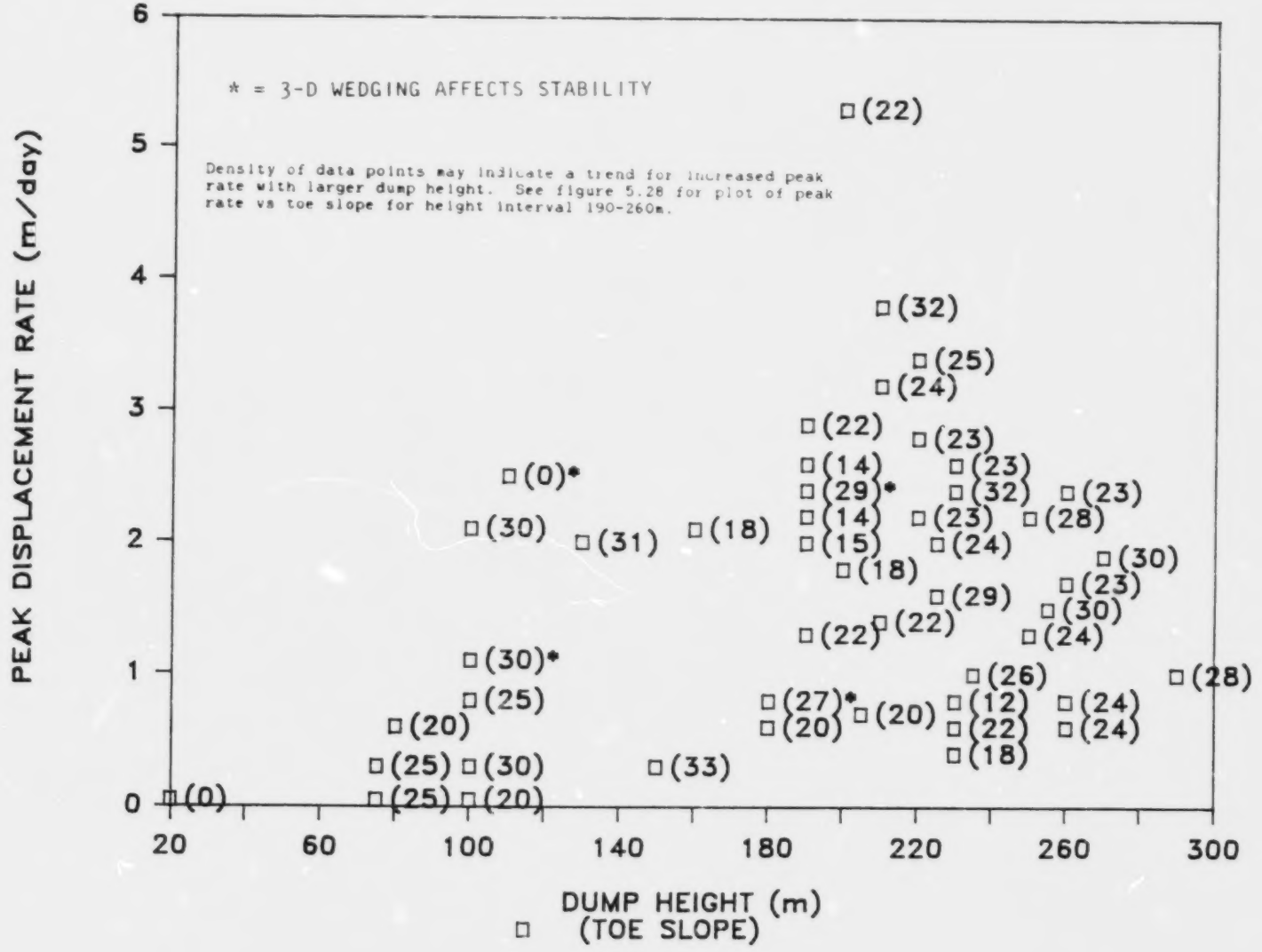

(X,Y)
X = RATE OF DUMPING (bcm/m/day)
Y = TOE SLOPE (deg)

BACKGROUND RATE OF CREST DISPLACEMENT vs DUMP HEIGHT

Figure 5.26

Golder Associates

PROJECT NO ..... DRAWN ..... REVIEWED ..... DATE .....



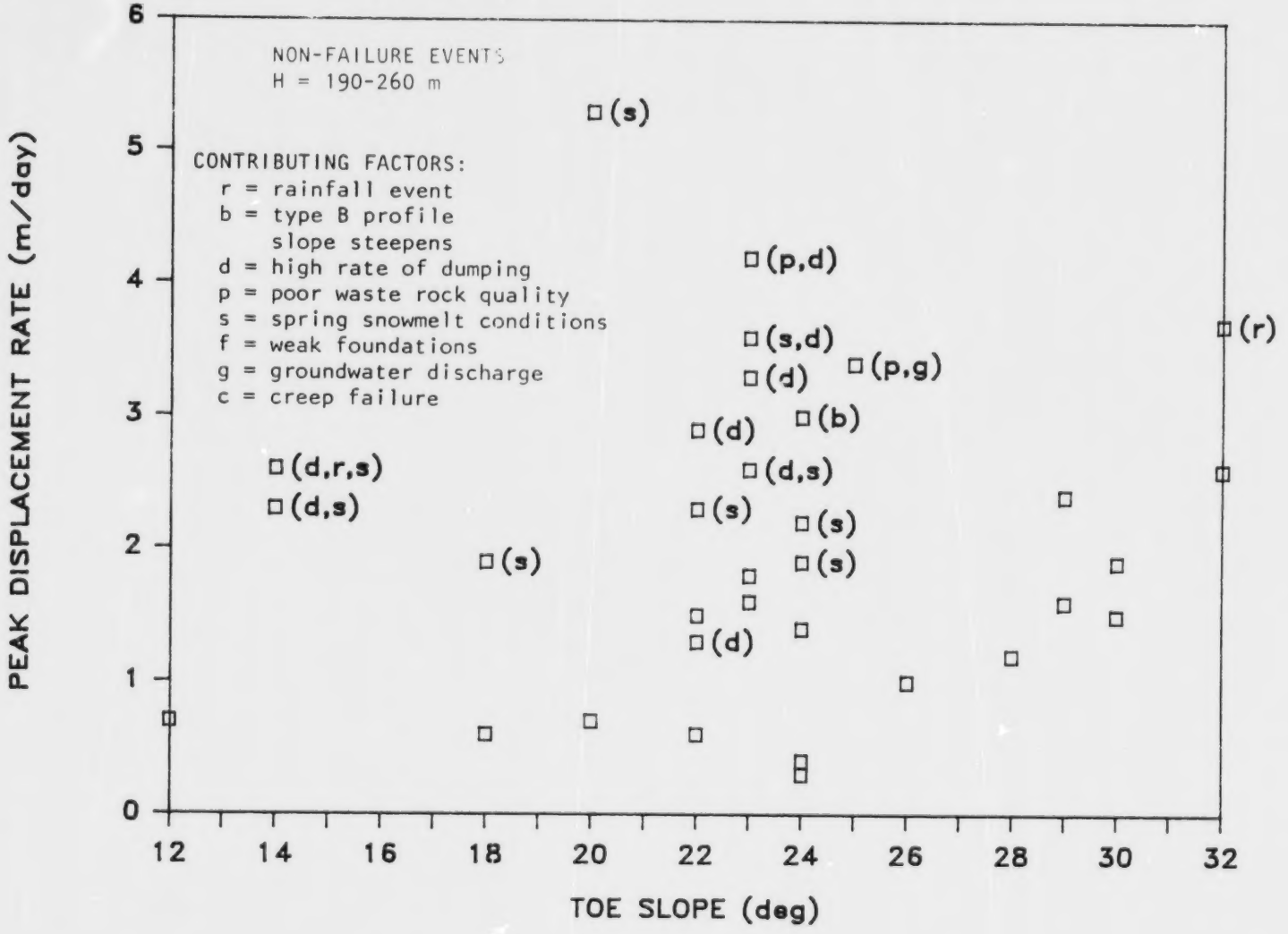

PEAK RATE OF CREST DISPLACEMENT vs DUMP HEIGHT FOR NON-FAILURE EVENTS

Figure 5.27

Golder Associates

PROJECT NO. ..... DRAWN ..... REVIEWED ..... DATE .....

Above about 25 deg the primary cause of high rate is steep slope. Identified causes shown in ( ) appear to result in higher rates. In particular spring thaw is the most frequently identified cause. Points without identified cause show a trend to increase with toe slope.




PEAK RATE OF CREST DISPLACEMENT vs TOE SLOPE FOR NON-FAILURE EVENTS

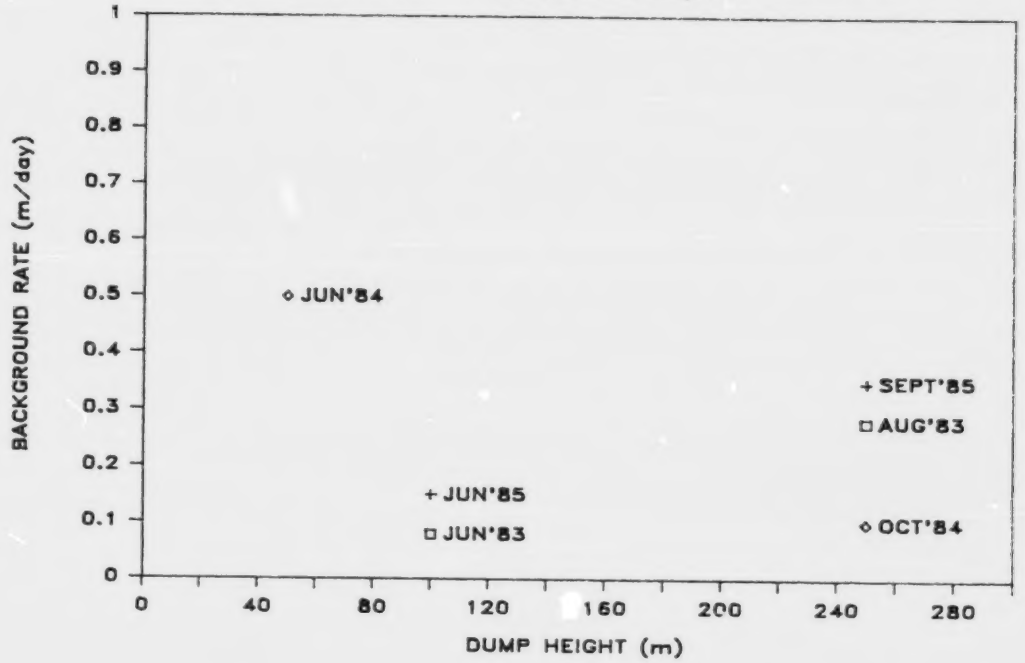
Figure 5.28

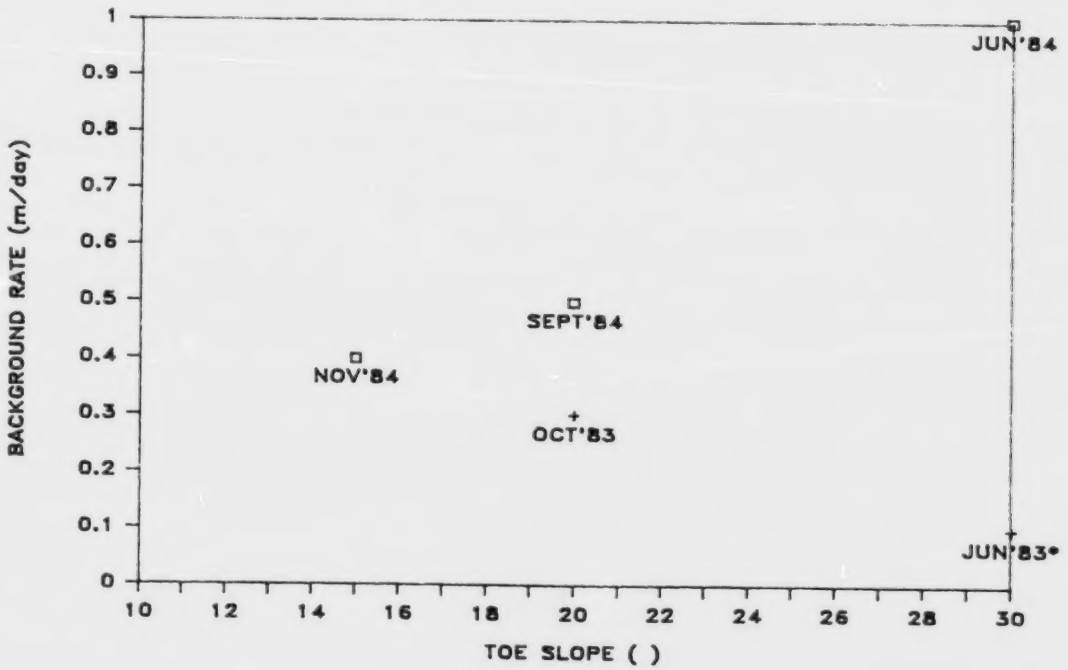
Golder Associates

BACKGROUND RATE OF CREST DISPLACEMENT
vs SLOPE and HEIGHT : SITE B

Figure 5.29

Favorable orientation of new dump platform appears to result in lower background rates in early stages when height is small, as in '83 and '85 cases. By contrast '84 example exhibits higher background rates when toe slope steep and crest orientation may be poor with respect to topogrgaphic v-shape.




PROJECT NO. ..... DRAWN ..... REVIEWED ..... DATE .....

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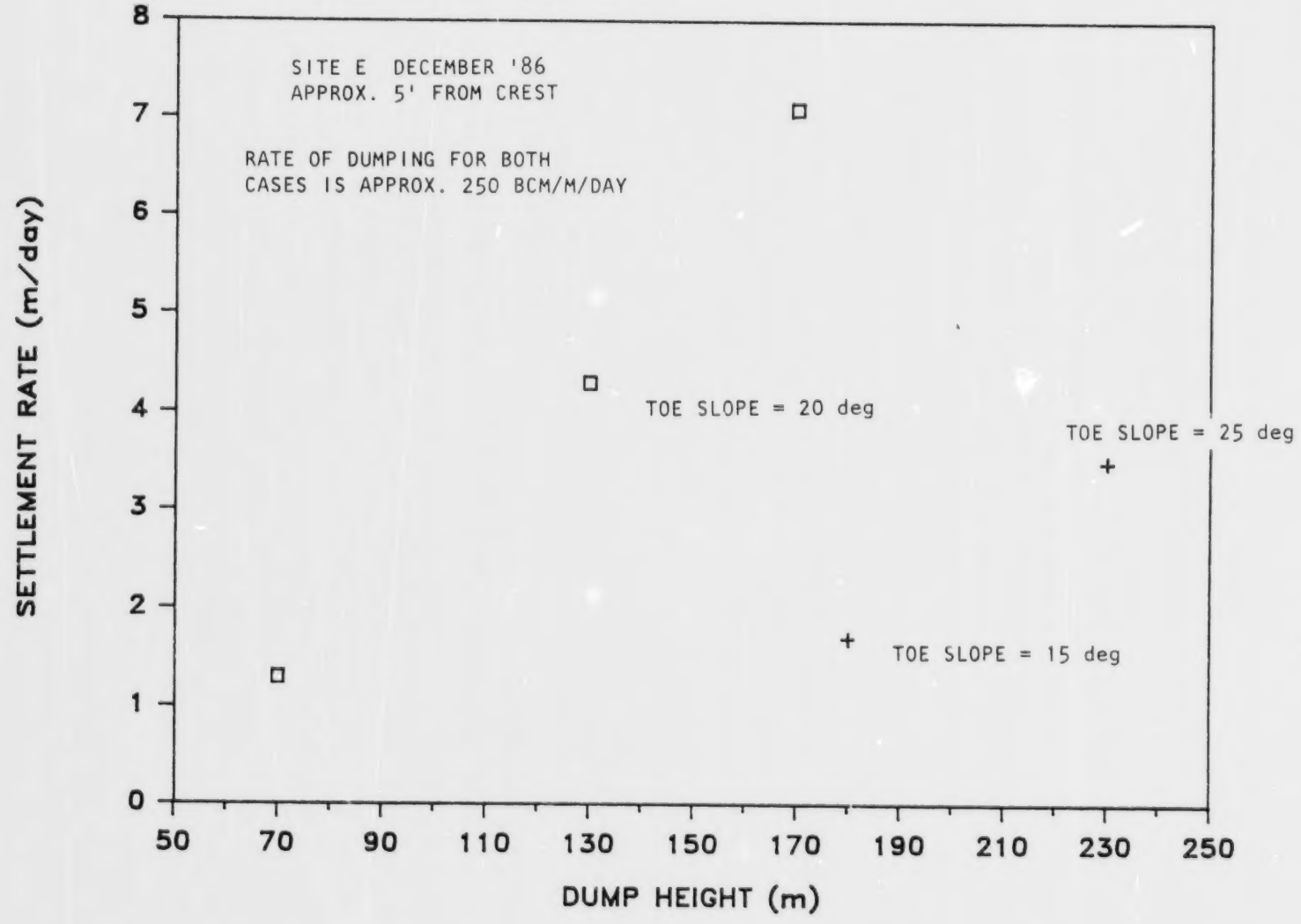
PROJECT NO. ..... DRAWN ..... REVIEWED ..... DATE .....

Data for two specific dumps shows a more clear increase in settlement rate for higher dumps. Note relatively high rates of both settlement and dumping.



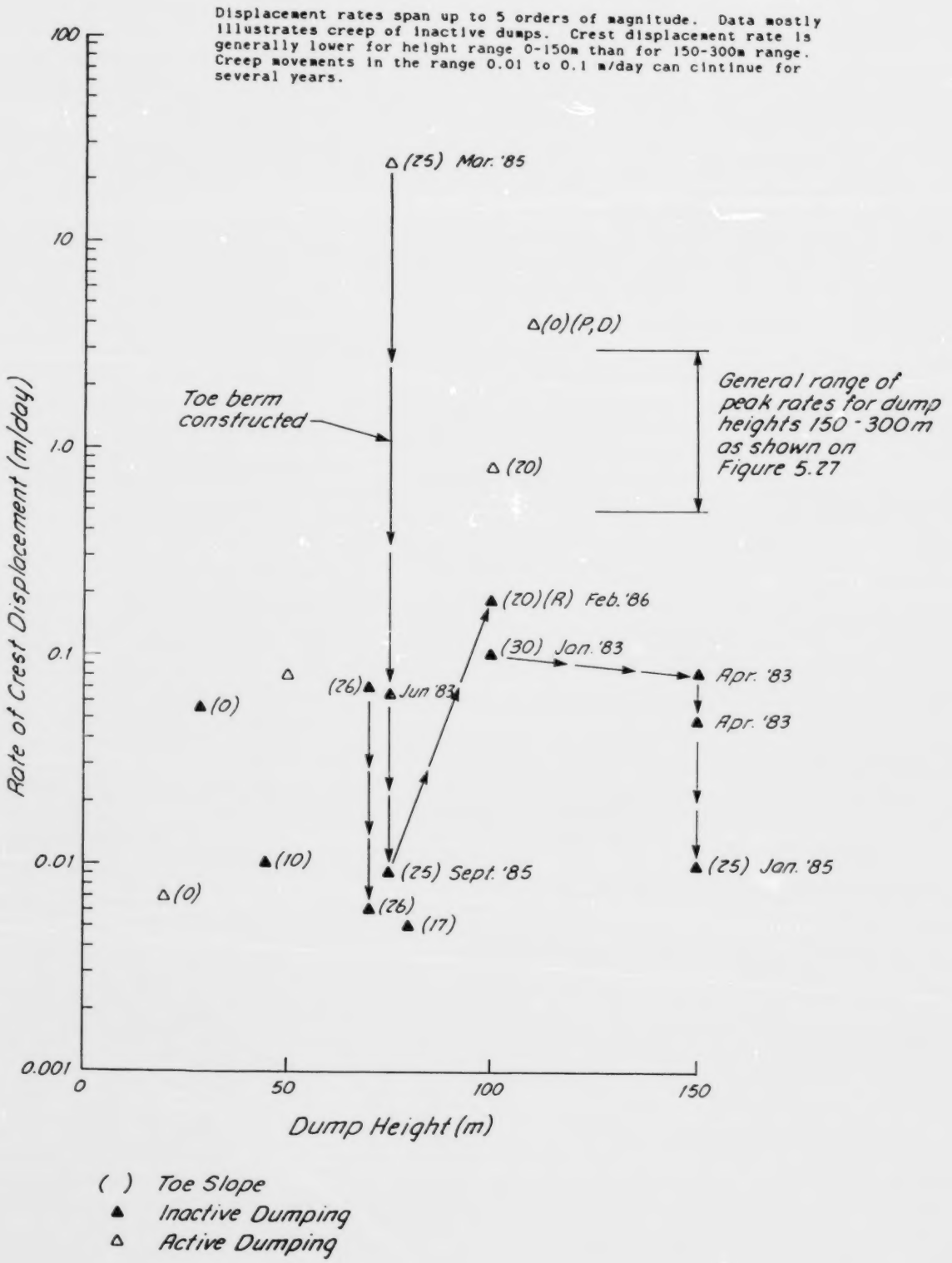

CREST SETTLEMENT RATE vs DUMP HEIGHT
SITE E

Figure 5.30

Golder Associates

RATE OF CREST DISPLACEMENT vs DUMP HEIGHT
0 TO 150M HEIGHT RANGE

Figure **5.31**

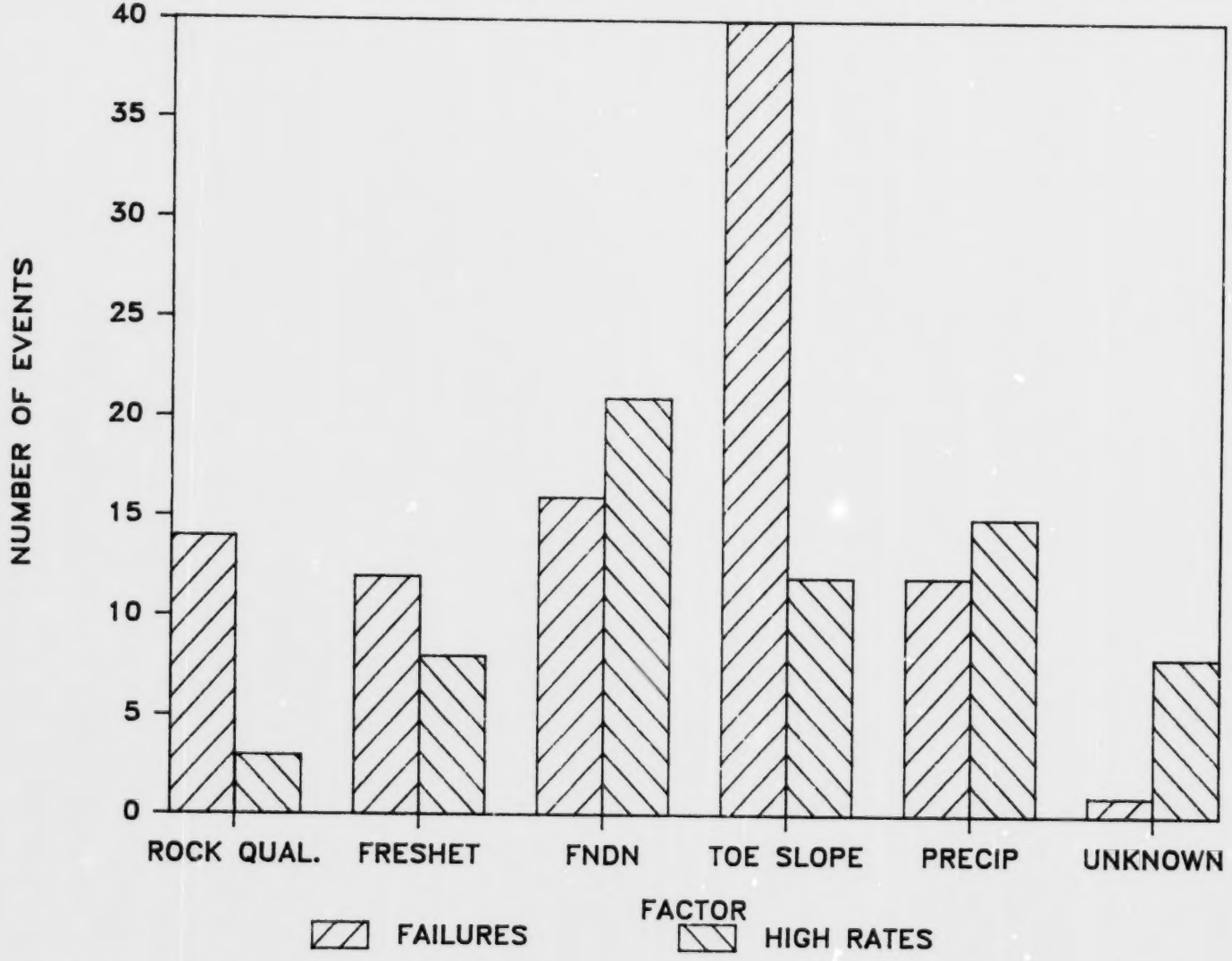



PROJECT NO. 862-1231E DRAWN BAD REVIEWED DATE May '87

Golder Associates

PROJECT NO. ..... DRAWN ..... REVIEWED ..... DATE .....

Steep toe slopes in excess of 25 deg are clearly the most frequent cause of failure. Poor rock quality, spring thaw, weak foundation conditions and high rainfall appear to rank approximately equally.




FREQUENCY OF CAUSATIVE FACTORS

Figure 5.32

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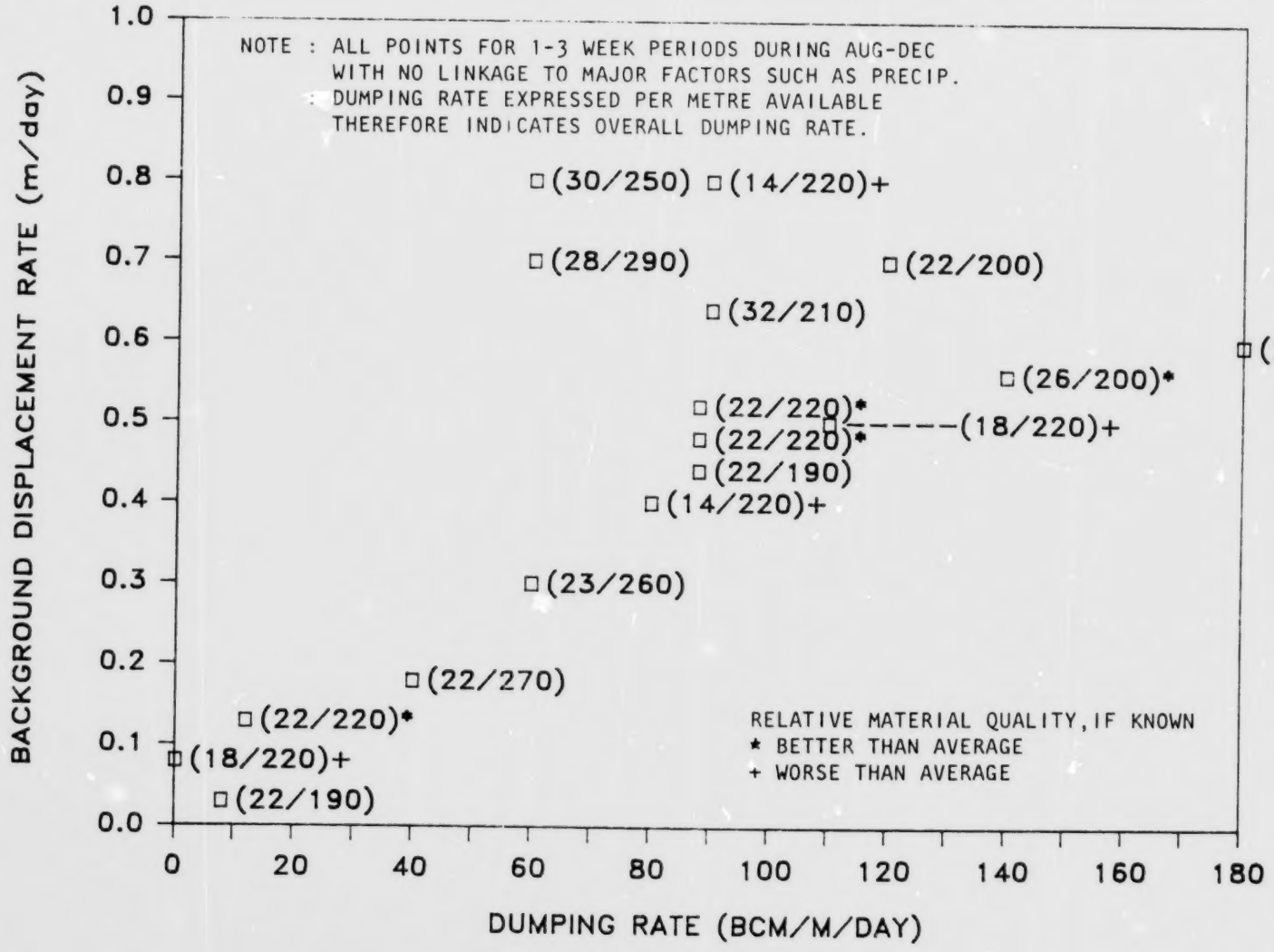
PROJECT NO. ..... DRAWN ..... REVIEWED ..... DATE .....

Except for extreme values of toe slope data indicates trend for increased background displacement rate with higher rates of dumping. Steepest and flattest toe slopes show highest background rates.




BACKGGROUND CREST DISPLACEMENT RATE vs RATE OF DUMPING

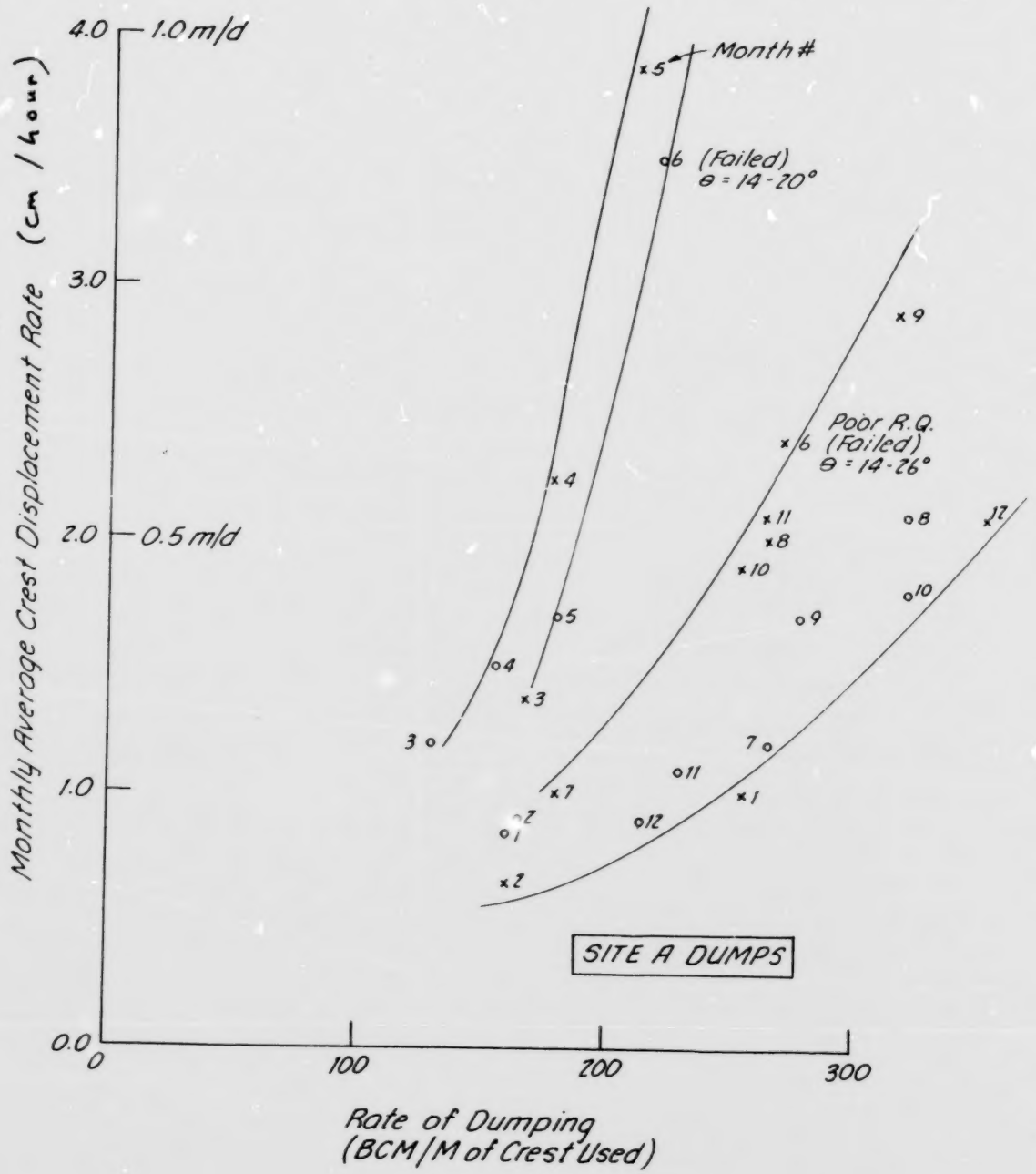
Figure 5.33

Golder Associates

# AVERAGE MONTHLY CREST DISPLACEMENT RATE VERSUS DUMPING RATE

Figure 5.34

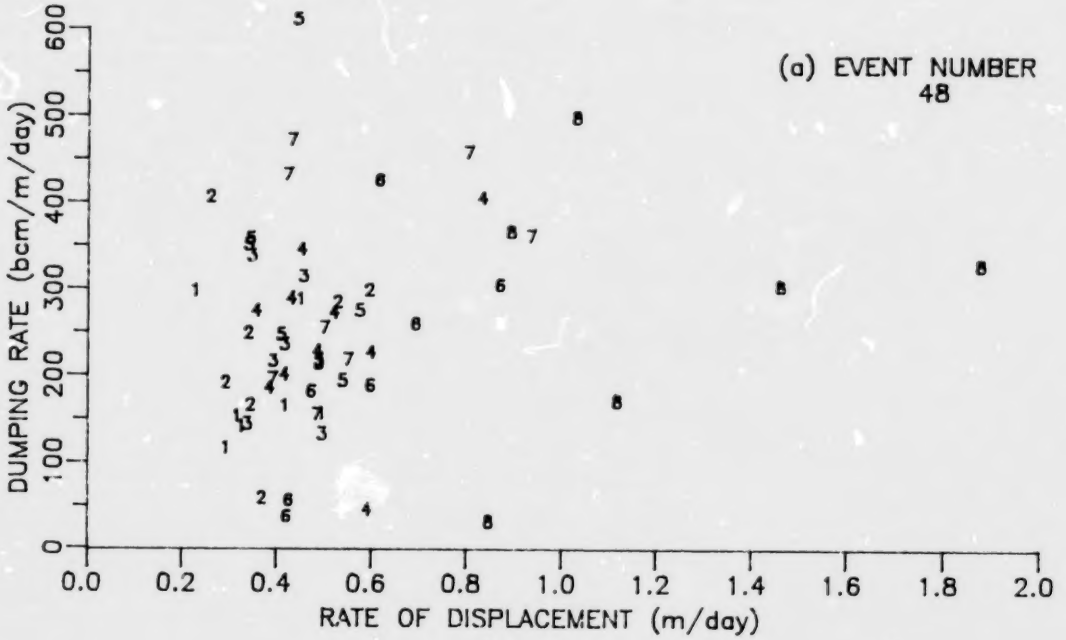
Monthly average displacement rate increases during late winter and spring following upper band. Remainder of year lies within distinctly lower band. Note that both dumps considered had low to moderate toe slopes.




PROJECT NO. 867-1631 DRAWN BAD REVIEWED DATE June 87

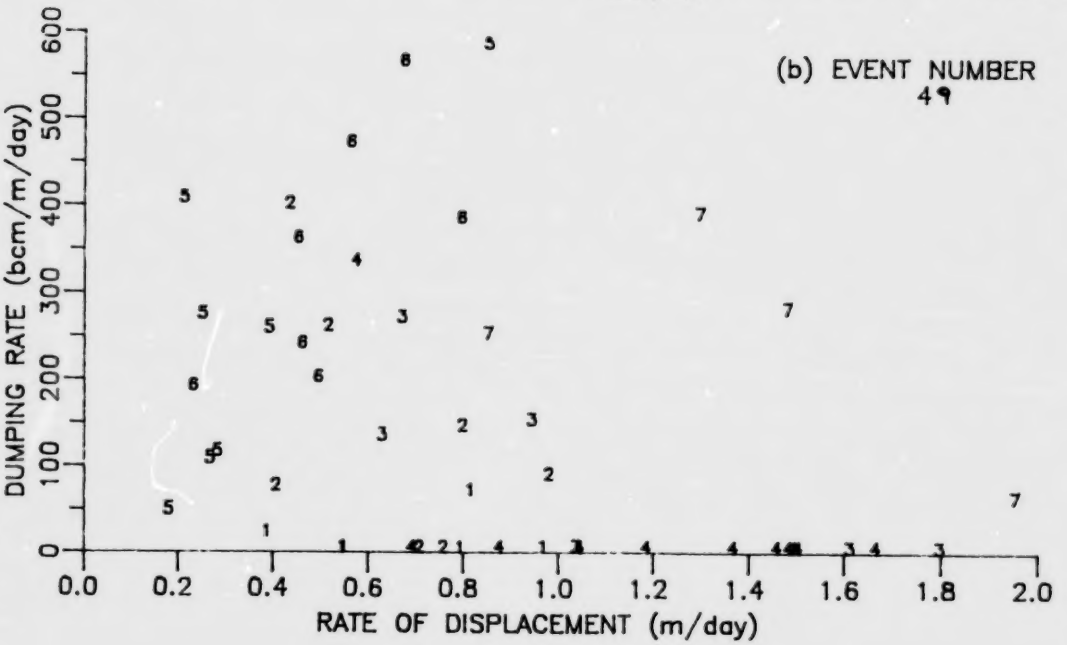
# DUMPING RATE vs CREST DISPLACEMENT RATE SITE A PRECEDING TWO FAILURE EVENTS

Figure 5.35




1 MAY 1–7
2 MAY 8–14
3 MAY 15–21
4 MAY 22–31
5 JUNE 1–7
6 JUNE 8–14
7 JUNE 15–21
8 JUNE 22–30

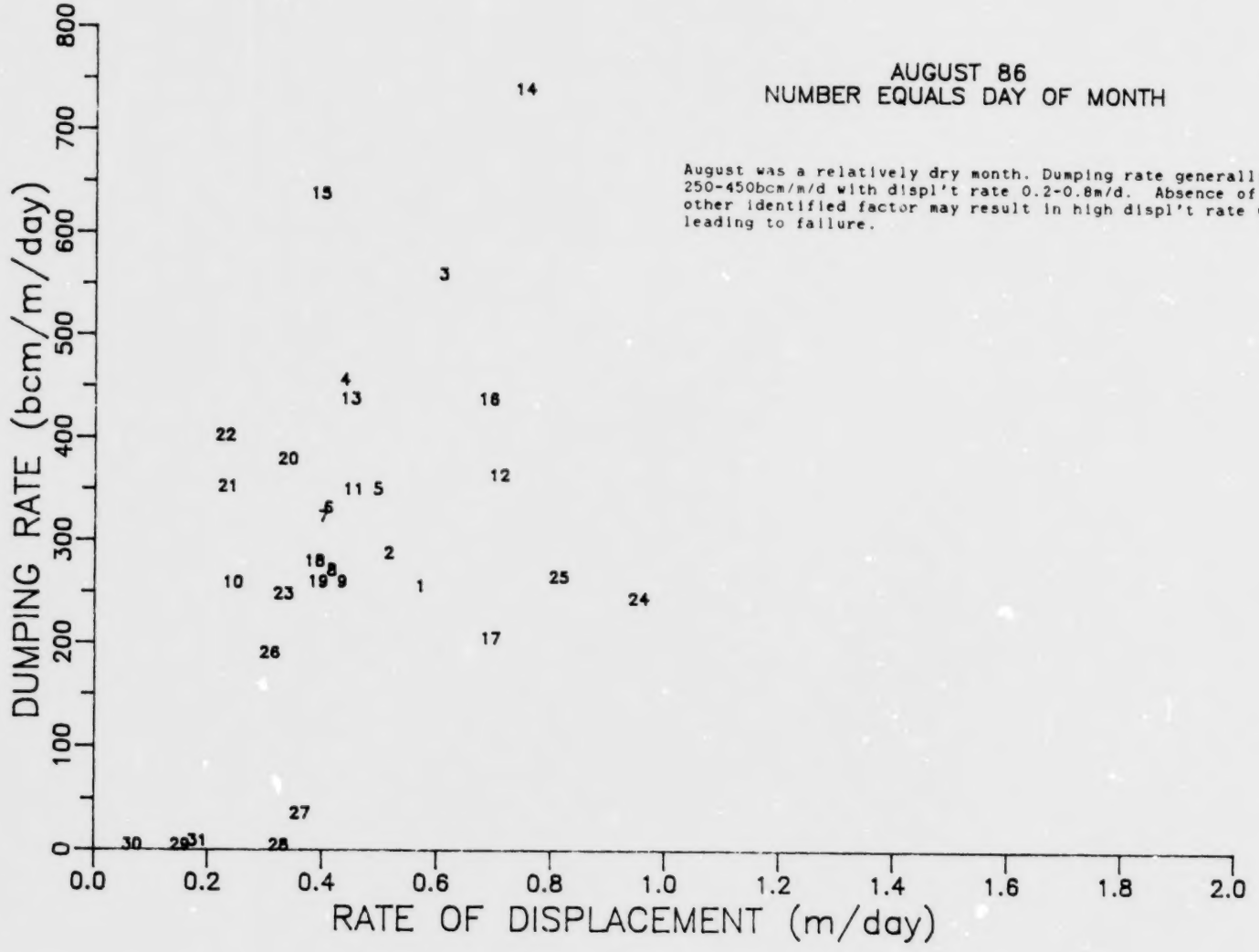
Plots characterise response in crest displacement to dumping activity. Upper plot shows that dumping rates of 200-300 BCM/metre/day in May resulted in crest displacement rates generally less than 0.6 m/day. Thereafter, increased dumping a effects of spring thaw resulted in increased crest displacemen rates to 0.6-1.2 m/day. Lower plot is for dump which appeared unstable over preceding 3 months. Frequency of dump short and push or shutdown was high. Failure at end of week 7 was prece by a month of relatively poor waste rock and by a major storm week prior.




PROJECT NO. ..... DRAWN ..... REVIEWED ..... DATE .....

Golder Associates

PROJECT NO. ..... DRAWN ..... REVIEWED ..... DATE .....

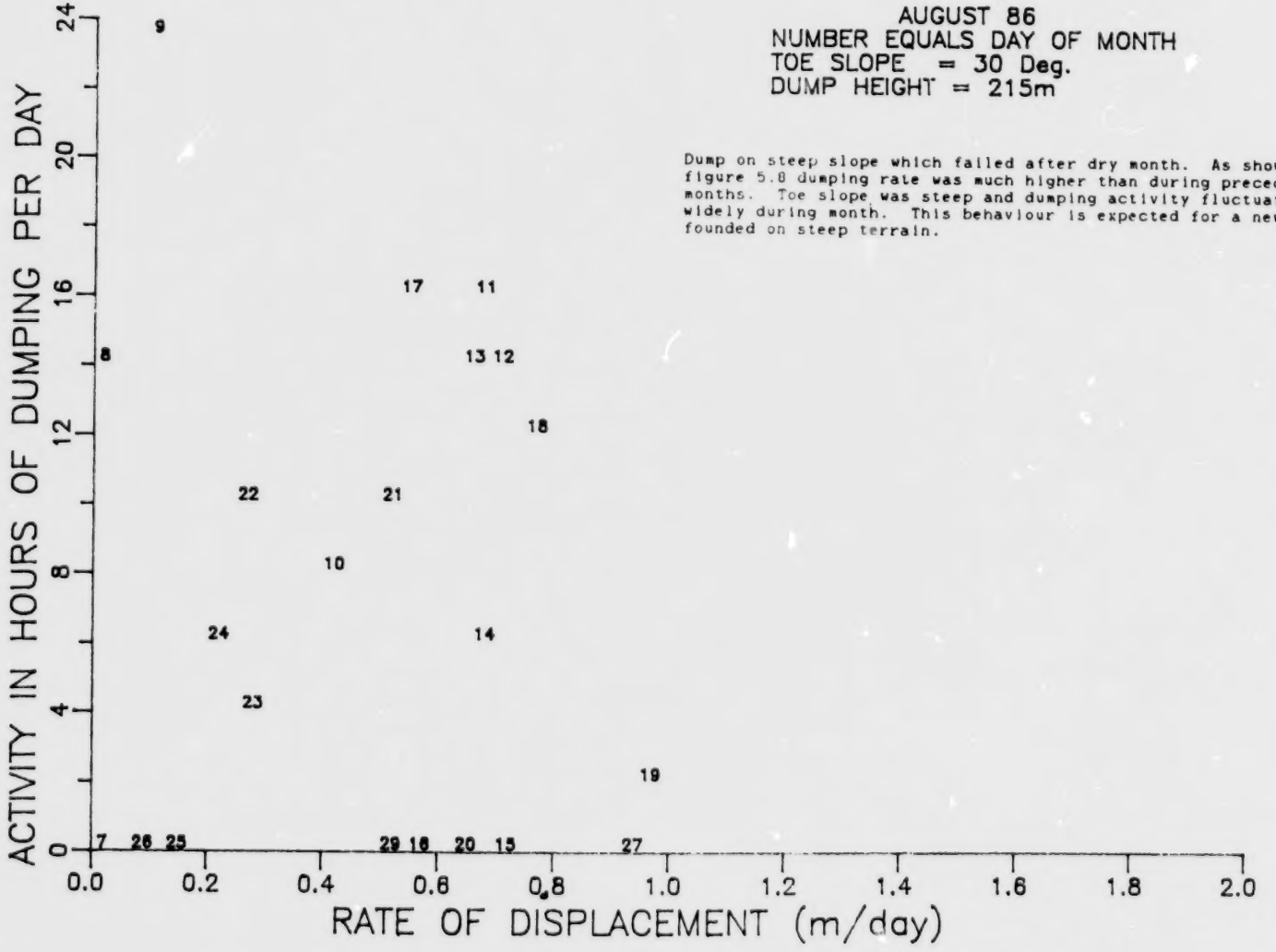



DUMPING RATE vs CREST DISPLACEMENT RATE
SITE A PRECEDING HIGH RATE EVENT

Figure 5.36

Golder Associates

PROJECT NO. ..... DRAWN ..... REVIEWED ..... DATE .....



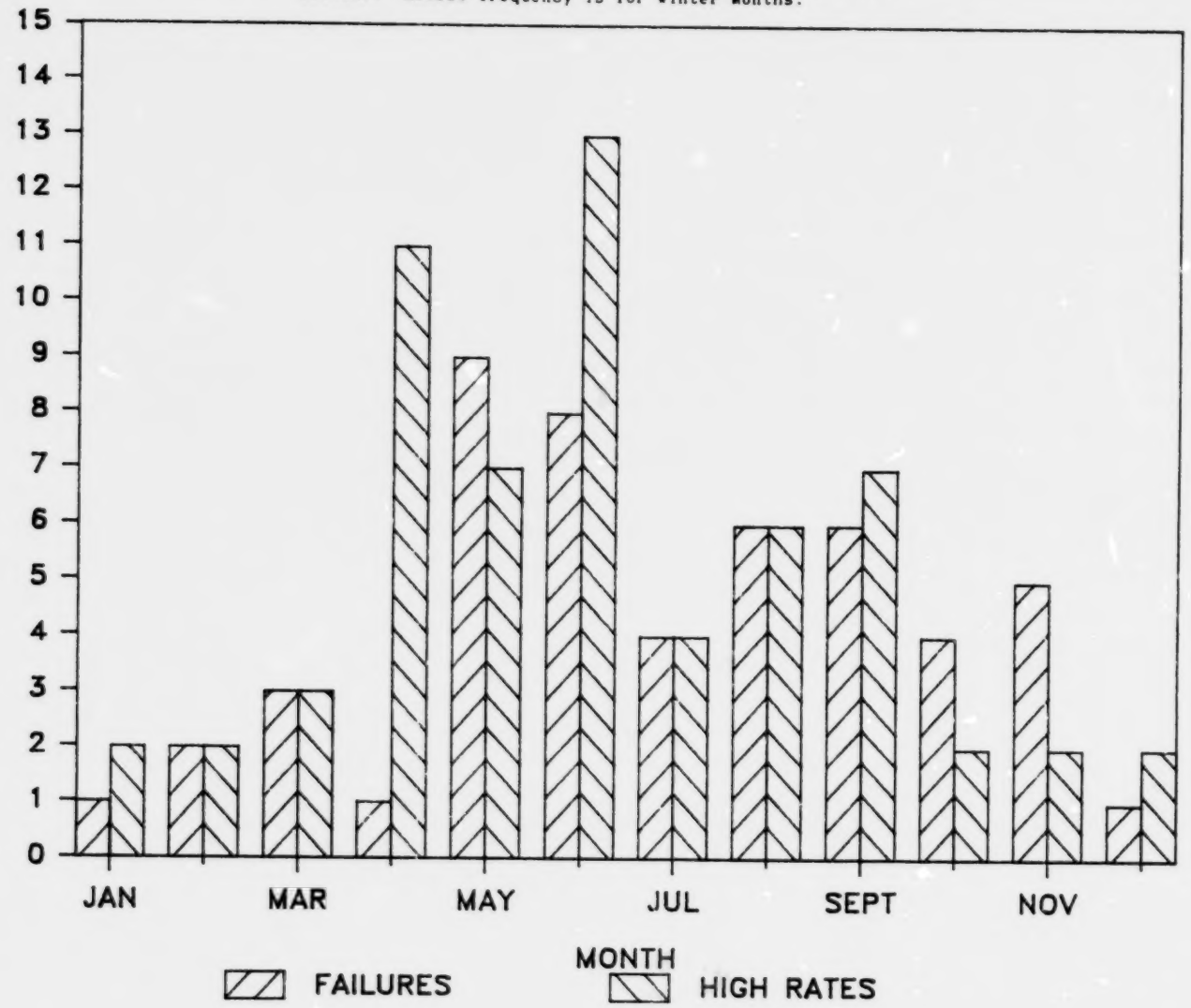

DUMPING ACTIVITY vs CREST DISPLACEMENT RATE
SITE C PRECEDING FAILURE EVENT

Figure 5.37

Golder Associates

PROJECT NO. ..... DRAWN ..... REVIEWED ..... DATE .....

Frequency of events is highest April - June corresponding to spring thaw. Rain storms and generally higher rates of dumping probably account for intermediate frequency during July - October. Lowest frequency is for winter months.

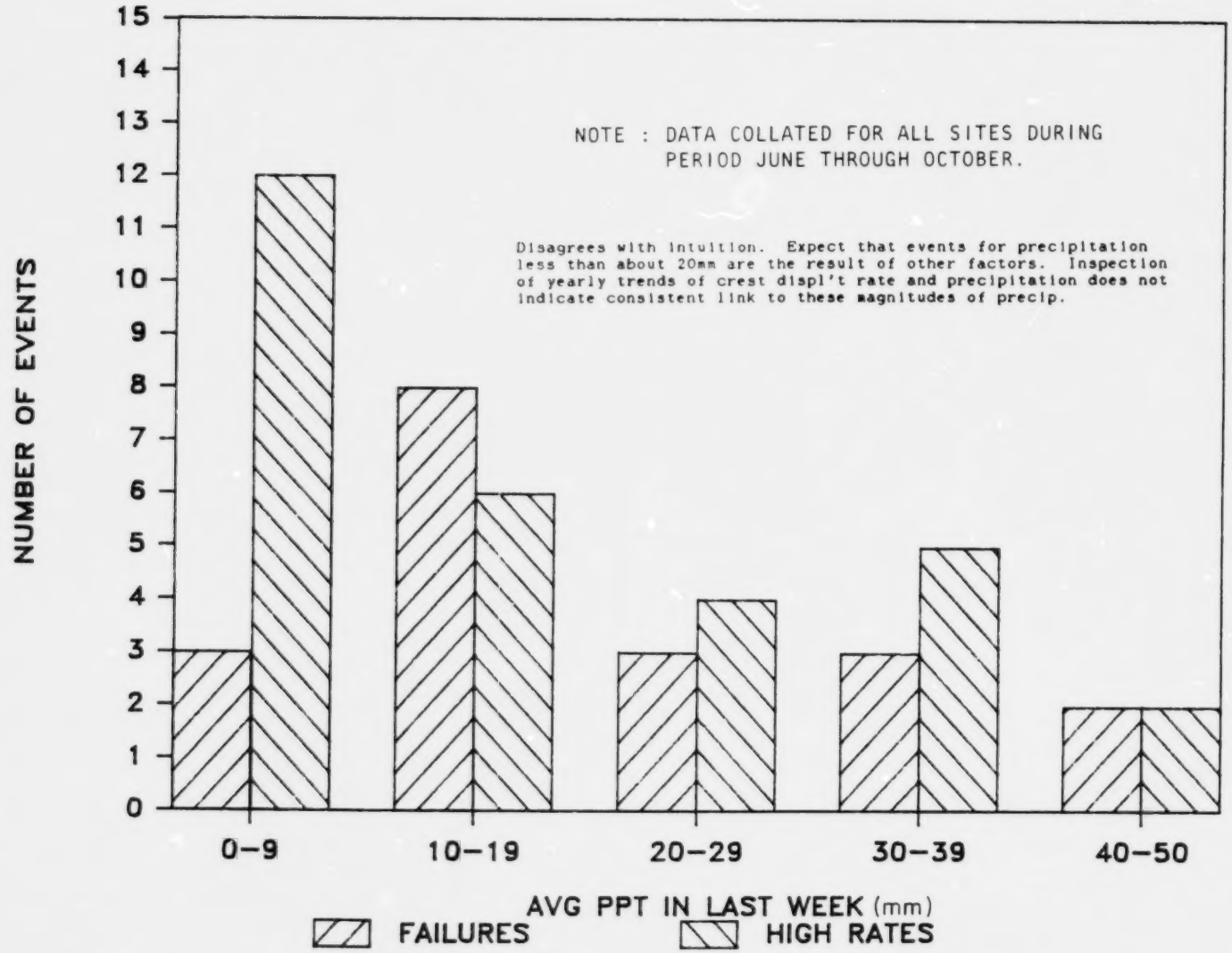



FREQUENCY OF FAILURE & HIGH RATE EVENTS WITH MONTH OF YEAR

Figure 5.38

Golder Associates

PROJECT NO. ..... DRAWN ..... REVIEWED ..... DATE .....



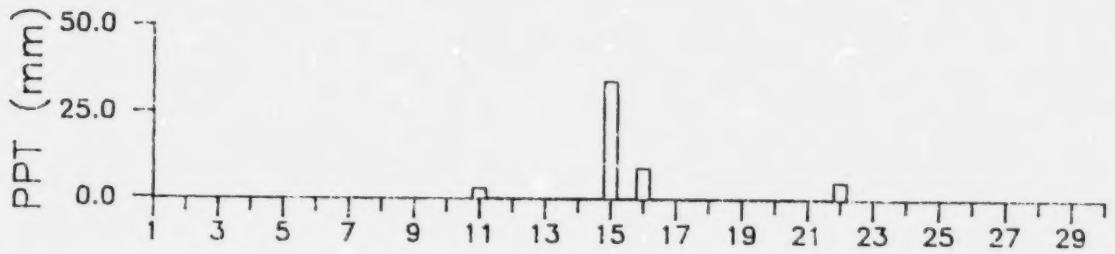

FREQUENCY OF FAILURE & HIGH RATE EVENTS
AVERAGE PRECIPITATION IN PRECEDING WEEK

Figure 5.39

Golder Associates

CREST DISPLACEMENT AND PRECIPITATION FOR SITE A DUMP FAILURE

Figure 5.40

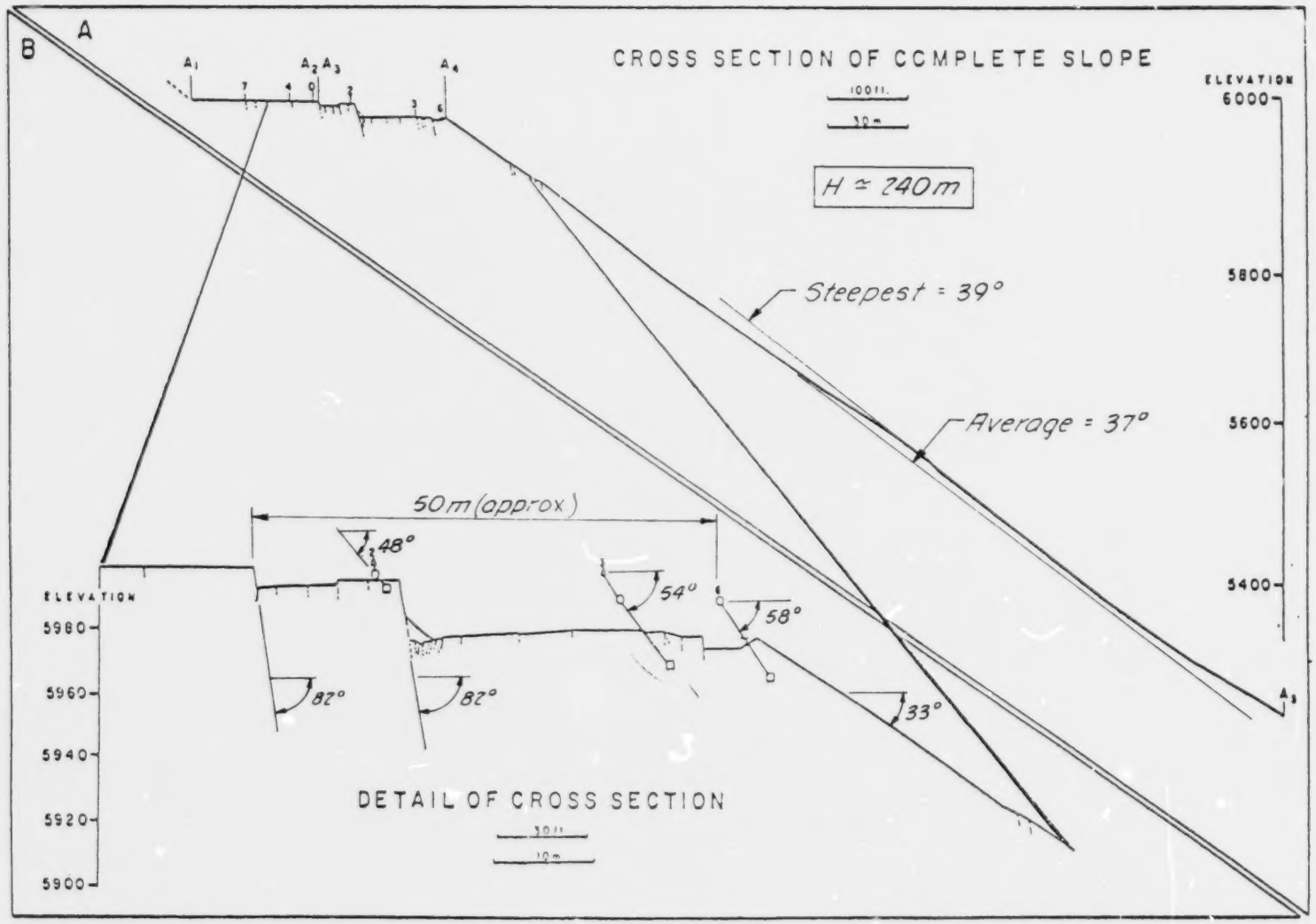



Failure may be due to combination of poor rock quality , spring thaw and nearly 50mm of rain in preceding week.

RATE OF DISPLACEMENT (m/day)
4.0
3.0
2.0
1.0
0.0
FROM FIGURE 5.2 JUNE 1-30
COMPLICATED BY CONSIGNMENT OF POOR QUALITY WASTE ROCK.
1 3 5 7 9 11 13 15 17 19 21 23 25 27 29
ELAPSED TIME (DAYS)

PROJECT NO. ..... DRAWN ..... REVIEWED ..... DATE .....

PROJECT NO. 862-1231E DRAWN - REVIEWED DATE June '87




Vectors of crest displacement are inclined at between 48 and 58 deg.. This agrees with results of model simulation and is typical of observations on many dump crests.

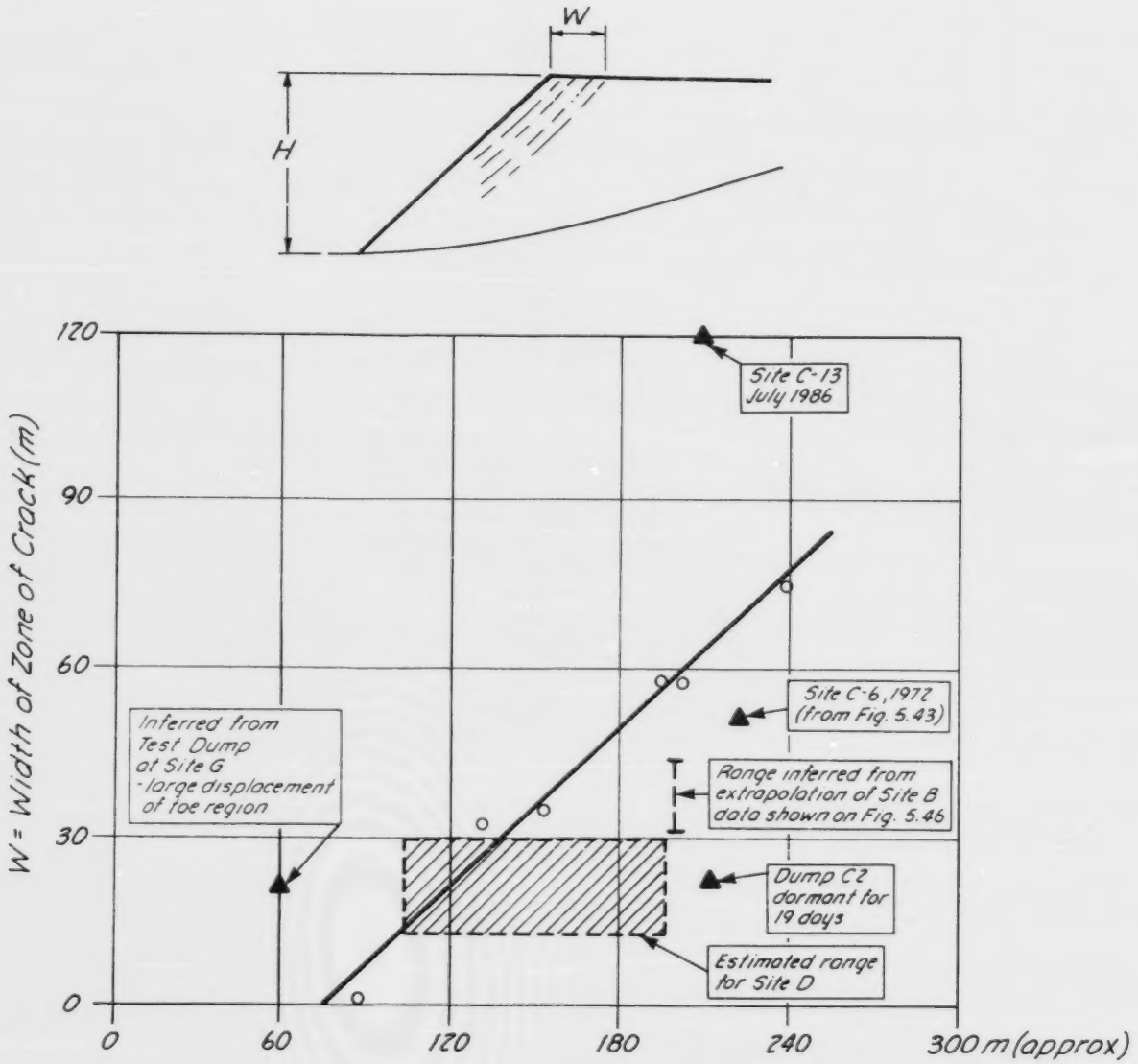
SURVEYED VECTORS OF CREST DISPLACEMENT
SITE C DUMP IN 1972

Figure 5.41

Golder Associates

# WIDTH OF ZONE OF CRACKING ADJACENT TO CREST OF WASTE DUMP

Figure **5.42**




NOTE:
Data from Kaiser Resources Ltd. & Fording Coal operations in southeastern B.C. (compiled by Golder Associates, 1979)

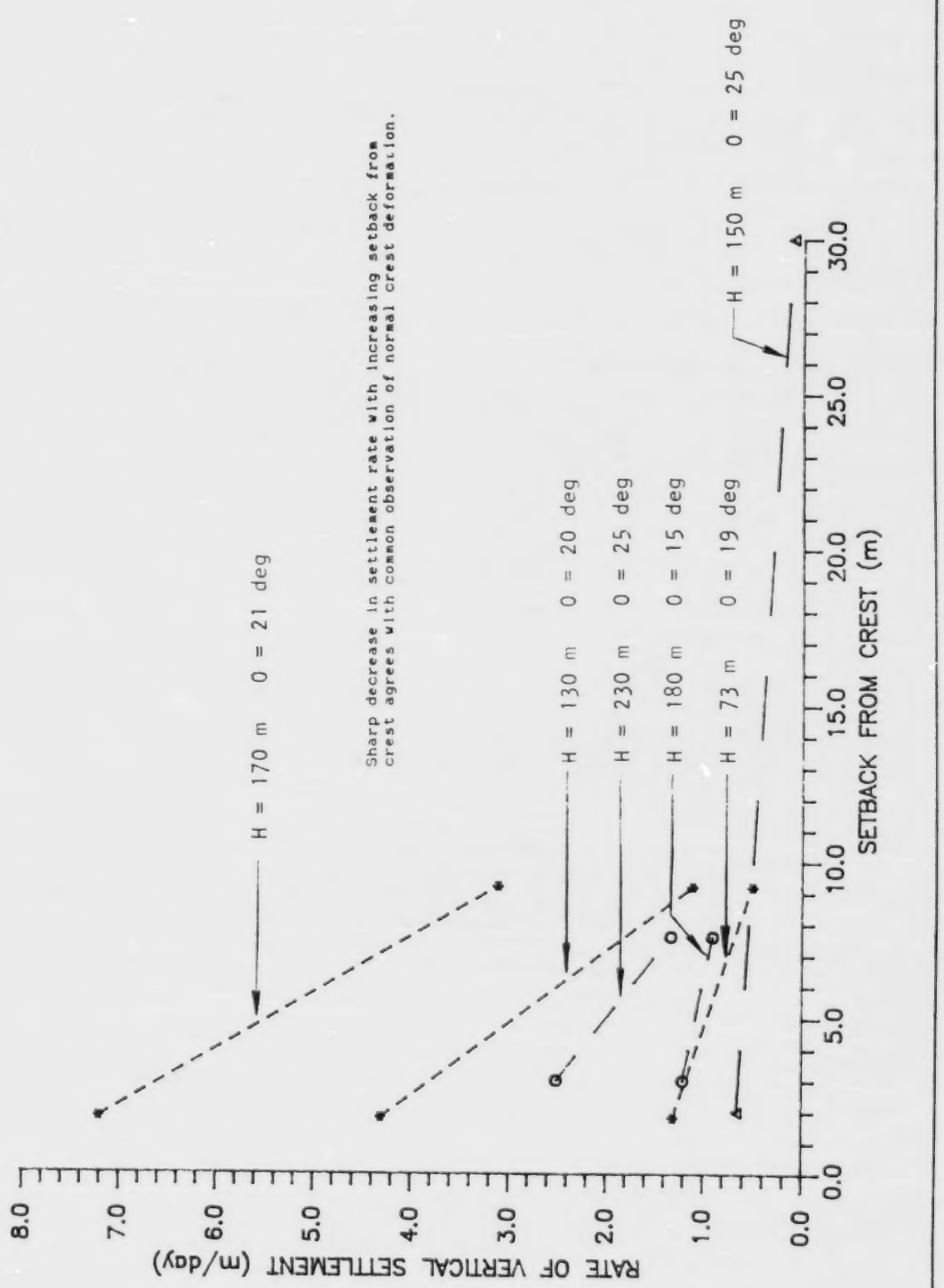
Original study of width of cracking has been supplemented additional data which mostly falls on or below the original trend. Results for points above trend are attributable to weak foundation conditions.

PROJECT NO. 861-1231.E DRAWN BAD REVIEWED ..... DATE May '87

PROJECT NO. ..... DRAWN ..... REVIEWED ..... DATE .....

VERTICAL SETTLEMENT RATE vs SETBACK FROM CREST

Figure 5.43

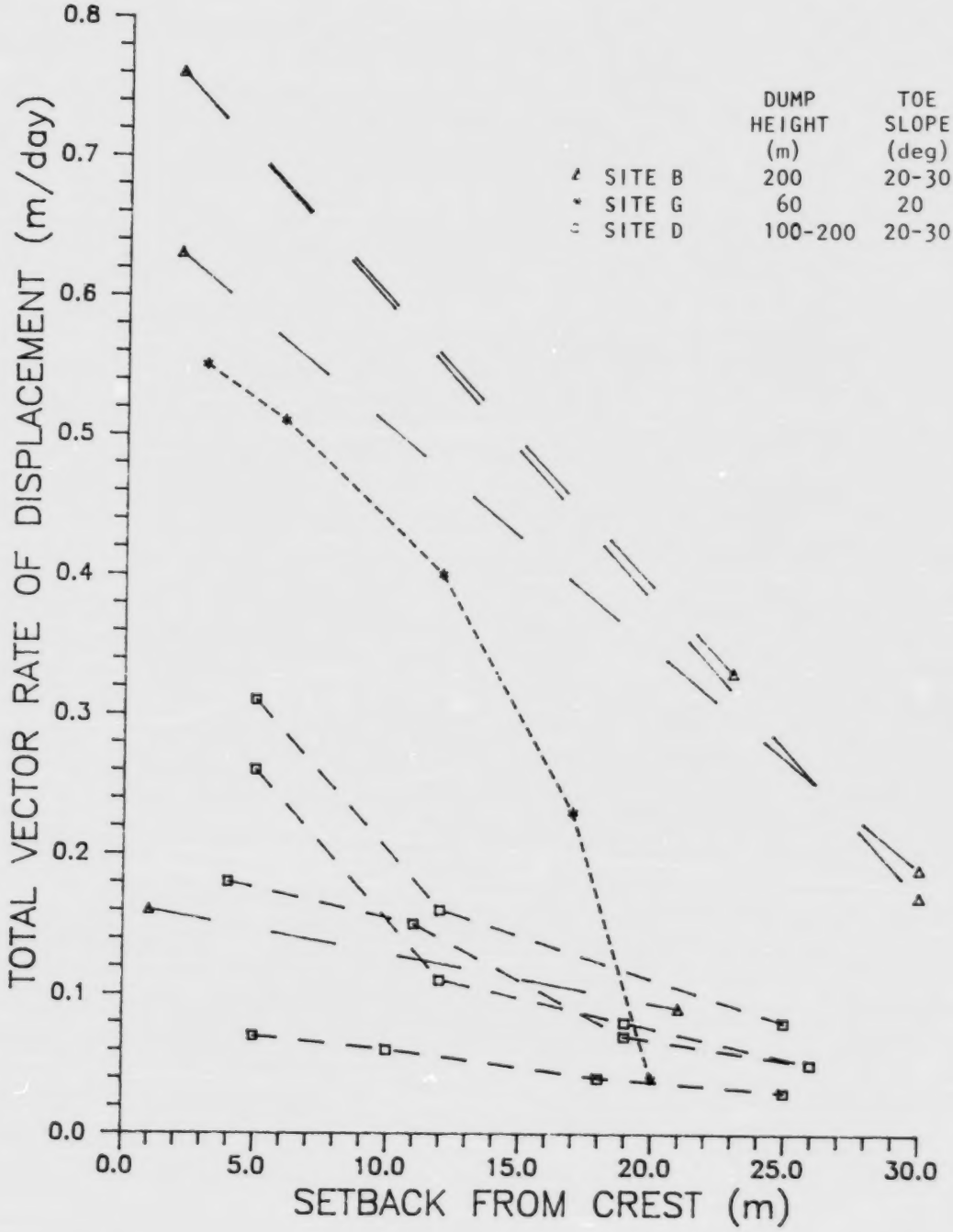



Golder Associates

TOTAL VECTOR DISPLACEMENT RATE
vs SETBACK FROM CREST

Figure 5.44

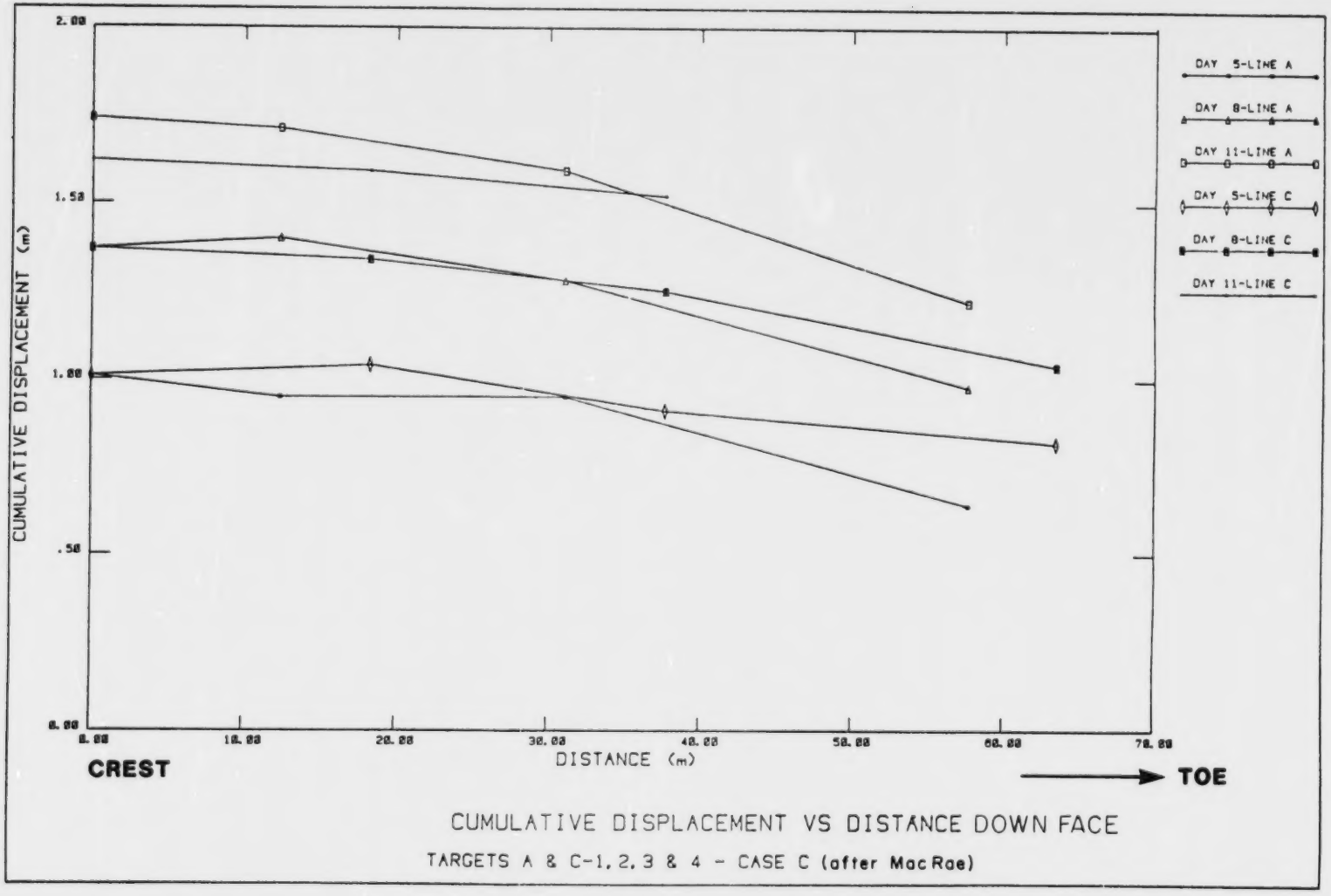
As on figure 5.43, rate decreases with setback from crest. Rate of decrease may be proportional displ't rate near crest. No clear trend with dump height or toe slope.




PROJECT NO. ..... DRAWN ..... REVIEWED ..... DATE .....

Golder Associates

PROJECT NO. 862-1231E DRAWN - REVIEWED DATE May '87




CUMULATIVE DISPLACEMENT VS DISTANCE DOWN FACE

TARGETS A & C-1, 2, 3 & 4 - CASE C (after MacRae)

Consistent trend with time for magnitude of total displacement to decrease with increasing distance down slope from face. Cumulative compressive strain below survey point should be a function of distance to toe.

CUMULATIVE DISPLACEMENT WITH DISTANCE DOWN DUMP FACE

Figure 5.45

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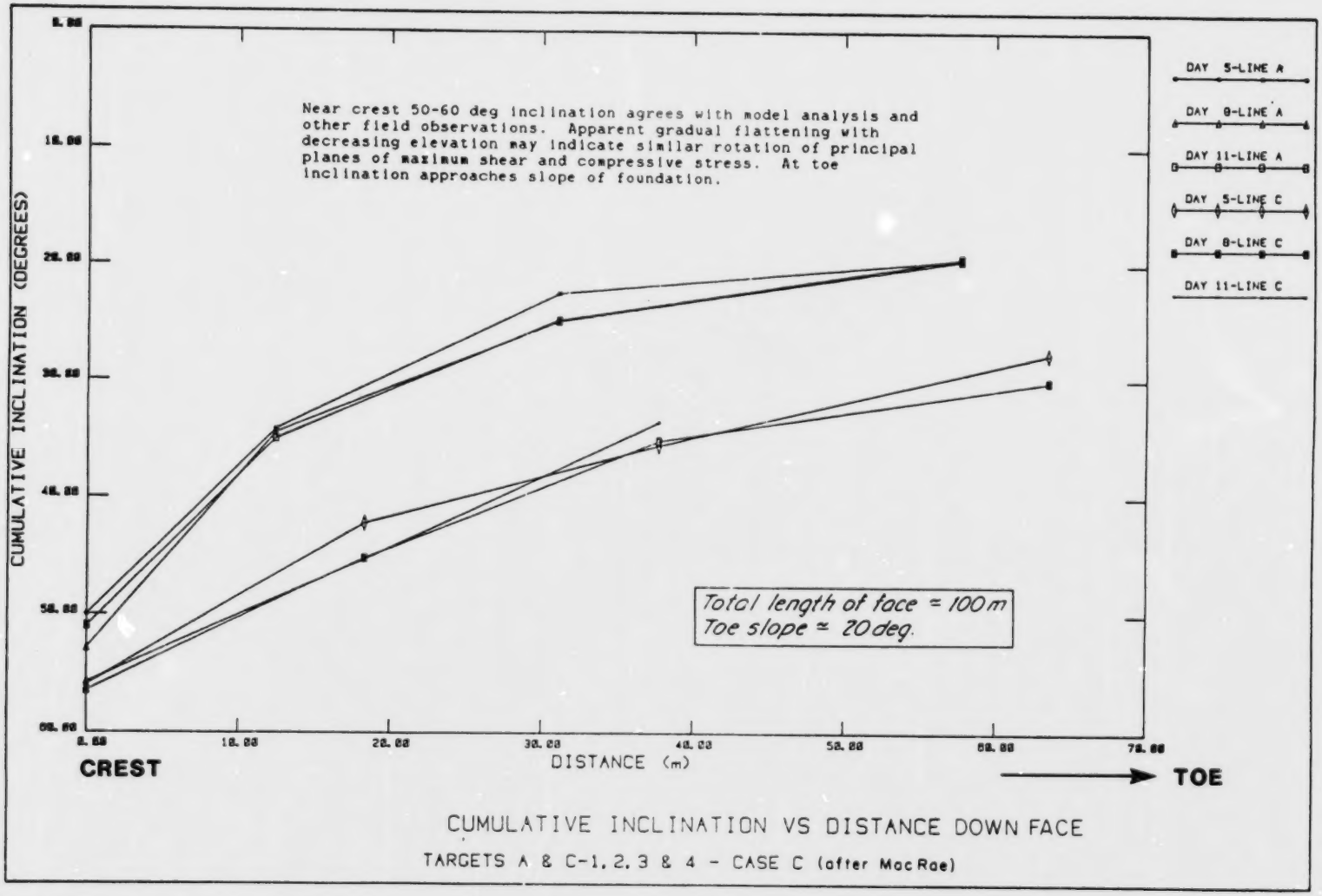
PROJECT NO. 862-1231E DRAWN - REVIEWED DATE June '87


Near crest 50-60 deg inclination agrees with model analysis and other field observations. Apparent gradual flattening with decreasing elevation may indicate similar rotation of principal planes of maximum shear and compressive stress. At toe inclination approaches slope of foundation.
DAY 5-LINE A
DAY 8-LINE A
DAY 11-LINE A
DAY 5-LINE C
DAY 8-LINE C
DAY 11-LINE C
Total length of face = 100m
Toe slope ≃ 20deg.
CUMULATIVE INCLINATION (DEGREES)
DISTANCE (m)
CREST
TOE

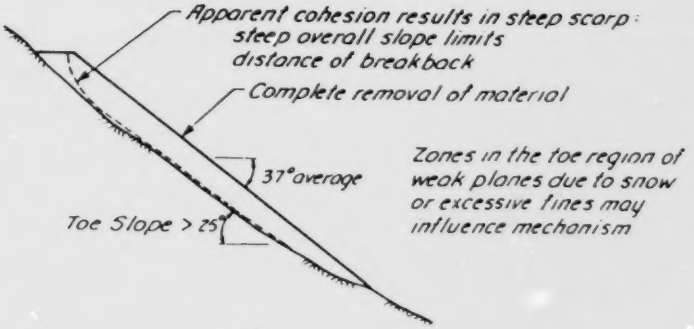
CUMULATIVE INCLINATION VS DISTANCE DOWN FACE

TARGETS A & C-1, 2, 3 & 4 - CASE C (after MacRae)

VECTOR INCLINATION vs DISTANCE DOWN DUMP FACE

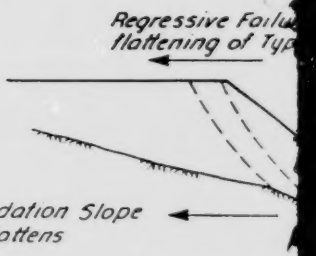
Figure 5.46

Golder Associates


Apparent cohesion results in steep scarp
steep overall slope limits
distance of breakback
Complete removal of material
37° average
Toe Slope > 25°
Zones in the toe region of
weak planes due to snow
or excessive fines may
influence mechanism

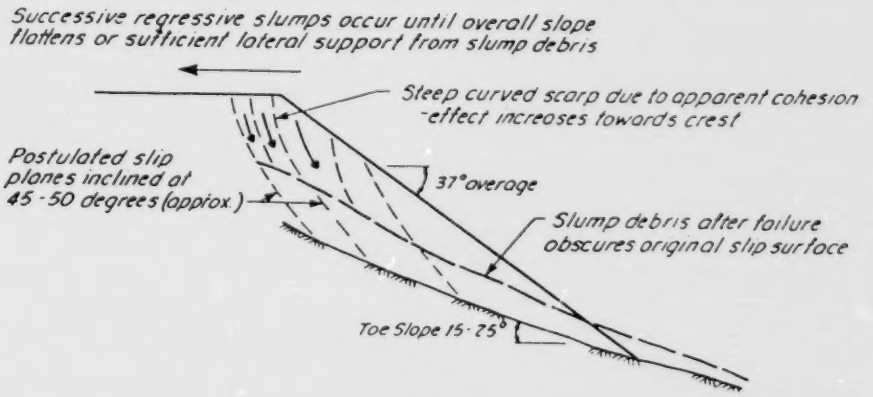
a)

**TYPE "A" FOUNDATION SLOPE PROFILE - STEEP SLOPES**



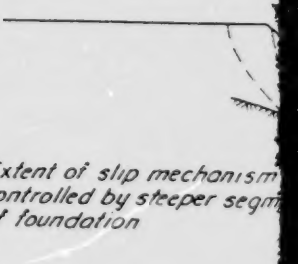

c)

**TYPE "B" FOUNDA**


Successive regressive slumps occur until overall slope
flattens or sufficient lateral support from slump debris
Steep curved scarp due to apparent cohesion
-effect increases towards crest
Postulated slip
planes inclined at
45 - 50 degrees (approx.)
37° average
Slump debris after failure
obscures original slip surface
Toe Slope 15 - 25°

b)

**TYPE "A" FOUNDATION SLOPE PROFILE - MODERATE SLOPES**



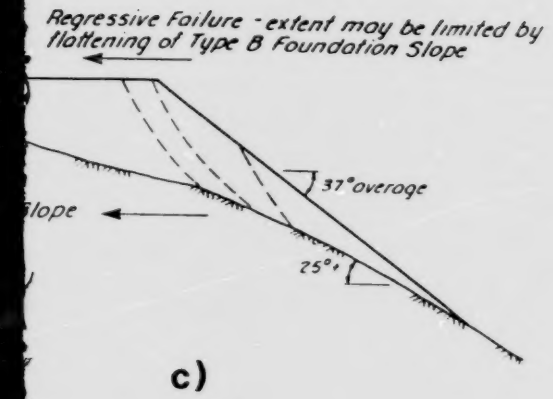

d)

**TYPE "C" FOUND**
**(i.e. composite of convex**

Sketches are intended to depict the range of situations for whic
mode and/or geometry of failure have been described. Emphasis i
placed on the complexity and progressive or staged nature of man
failures. The final configuration may bear little resemblance t
the iniating mechanism.

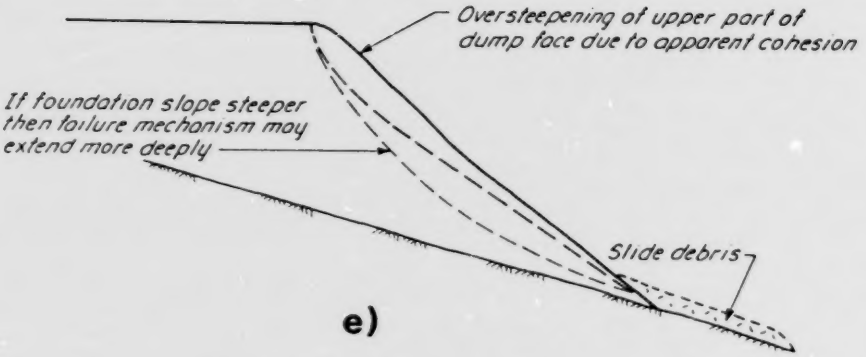
INFERRED FAILURE MECHANISMS

Figure 5.47



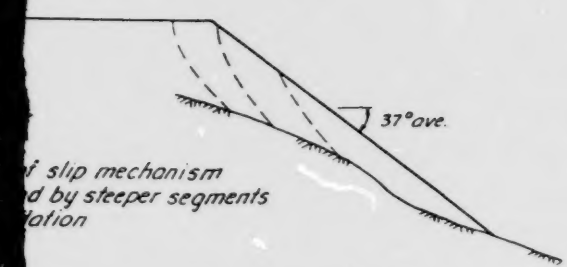

c)

"B" FOUNDATION SLOPE PROFILE


Oversteepening of upper part of dump face due to apparent cohesion
If foundation slope steeper then failure mechanism may extend more deeply
Slide debris

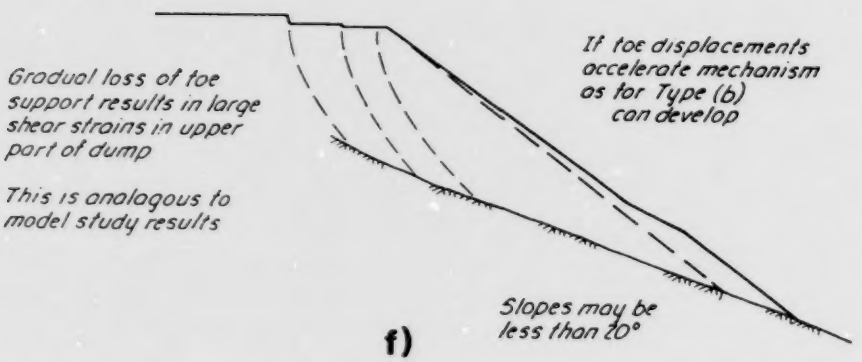
e)

CREST OVERSTEEP




d)

"C" FOUNDATION SLOPE
of convex and concave profiles)


Gradual loss of toe support results in large shear strains in upper part of dump
This is analogous to model study results
If toe displacements accelerate mechanism as for Type (b) can develop
Slopes may be less than 20°

f)

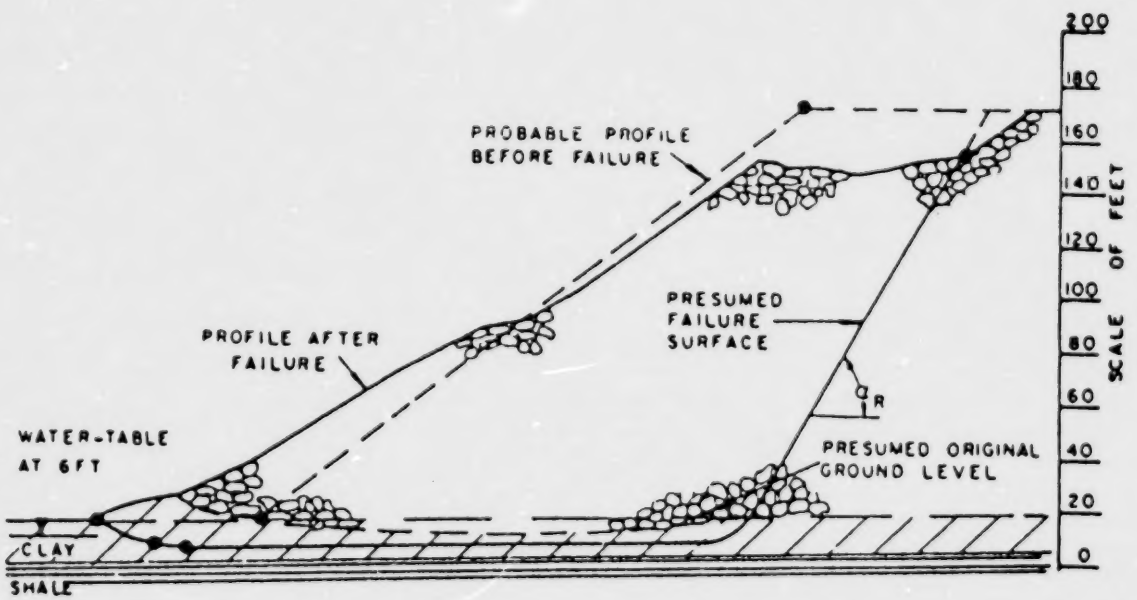
CREEP FAILURE

ns for which
Emphasis is
ture of many
semblance to

OBSERVED WEDGE MECHANISM FOR WASTE DUMP FAILURE ON WEAK FOUNDATION (Blight)

Figure **6.1**

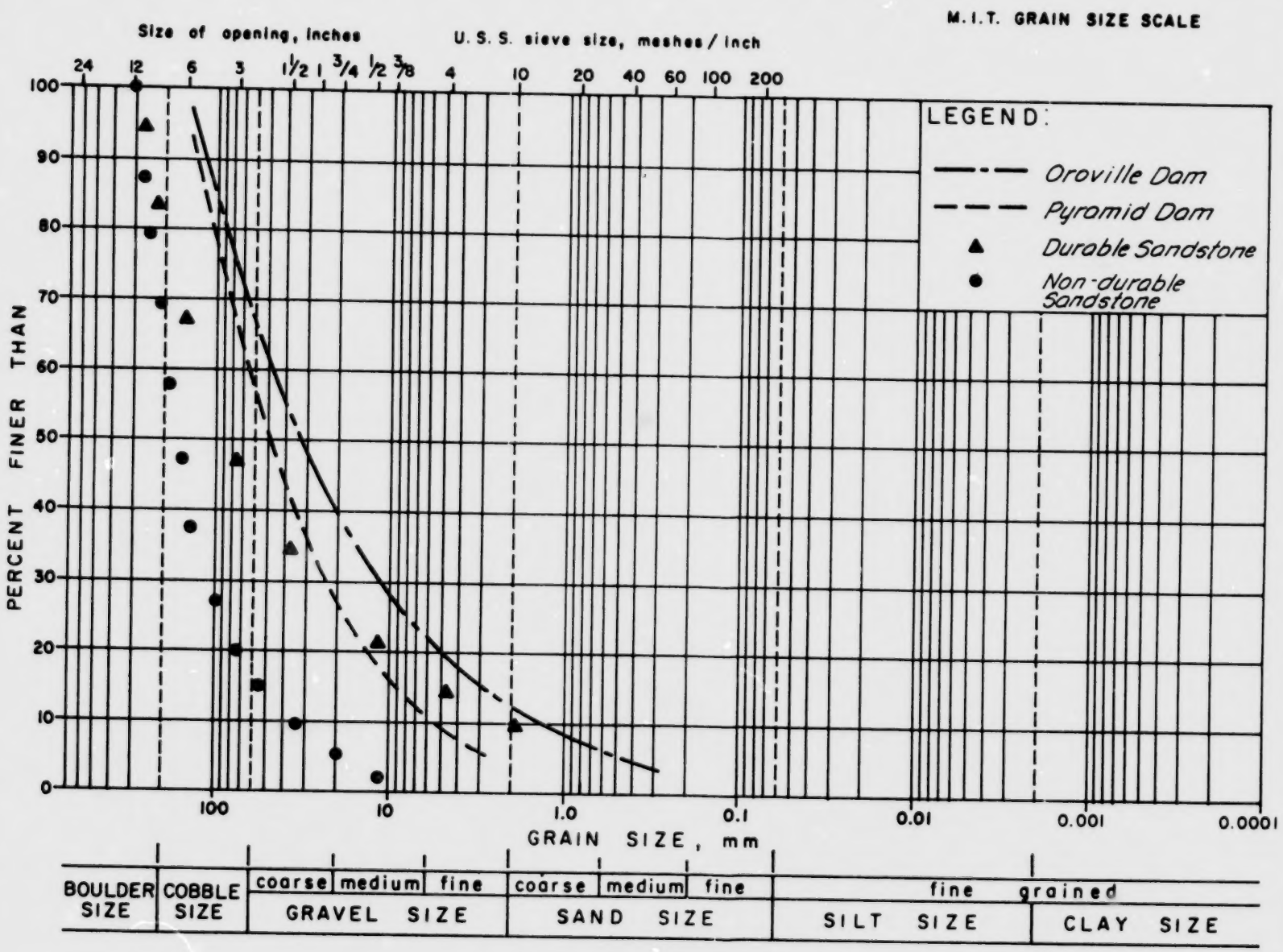
FIG. 3.—TYPICAL SLIDE IN A WASTE ROCK DUMP. PHOTOGRAPH AT TOP OF SLIDE WITH OVERLAY




DETAILS OF FIRST VLAKFONTEIN SLIDE

PROJECT NO. ..... DRAWN ..... REVIEWED ..... DATE .....

PROJECT NO. 867-1231E DRAWN REVIEWED DATE June '87



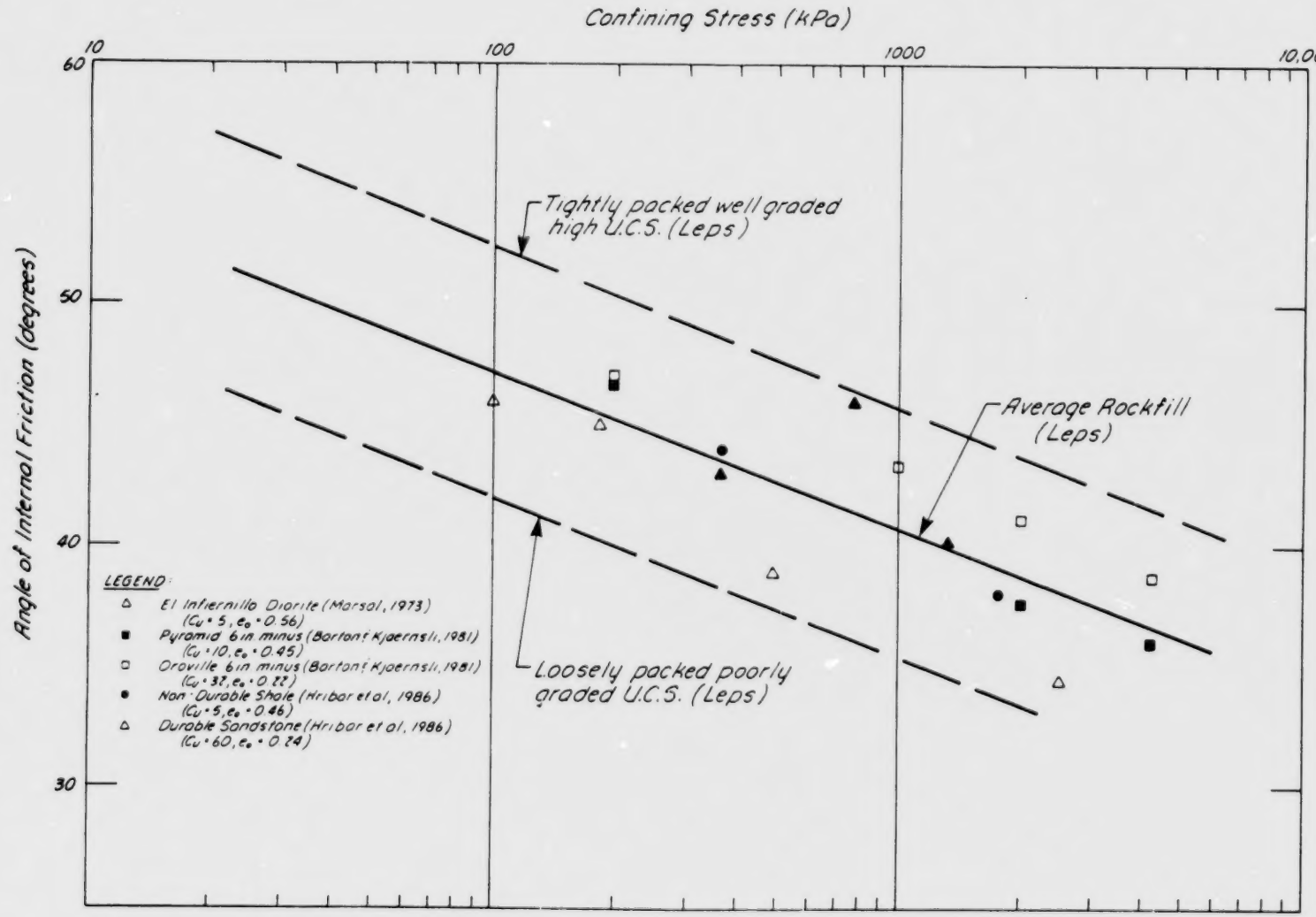

**GRAIN SIZE DISTRIBUTION**

ROCKFILL STRENGTH TEST SAMPLES

**Figure 6.2**

Golder Associates

PROJECT NO. 862-1231E DRAWN BAD REVIEWED DATE June '87


Confining Stress (kPa)
10
100
1000
10,000
60
50
40
30
Angle of Internal Friction (degrees)
Tightly packed well graded high U.C.S. (Leps)
Average Rockfill (Leps)
Loosely packed poorly graded U.C.S. (Leps)
LEGEND:
El Infiernillo Diorite (Marsal, 1973) (Cu = 5, e0 = 0.56)
Pyramid 6 in minus (Barton & Kjaernsli, 1981) (Cu = 10, e0 = 0.45)
Oroville 6 in minus (Barton & Kjaernsli, 1981) (Cu = 32, e0 = 0.22)
Non-Durable Shale (Hribar et al, 1986) (Cu = 5, e0 = 0.46)
Durable Sandstone (Hribar et al, 1986) (Cu = 60, e0 = 0.24)

ANGLE OF INTERNAL FRICTION WITH CONFINING STRESS

Figure 6.3

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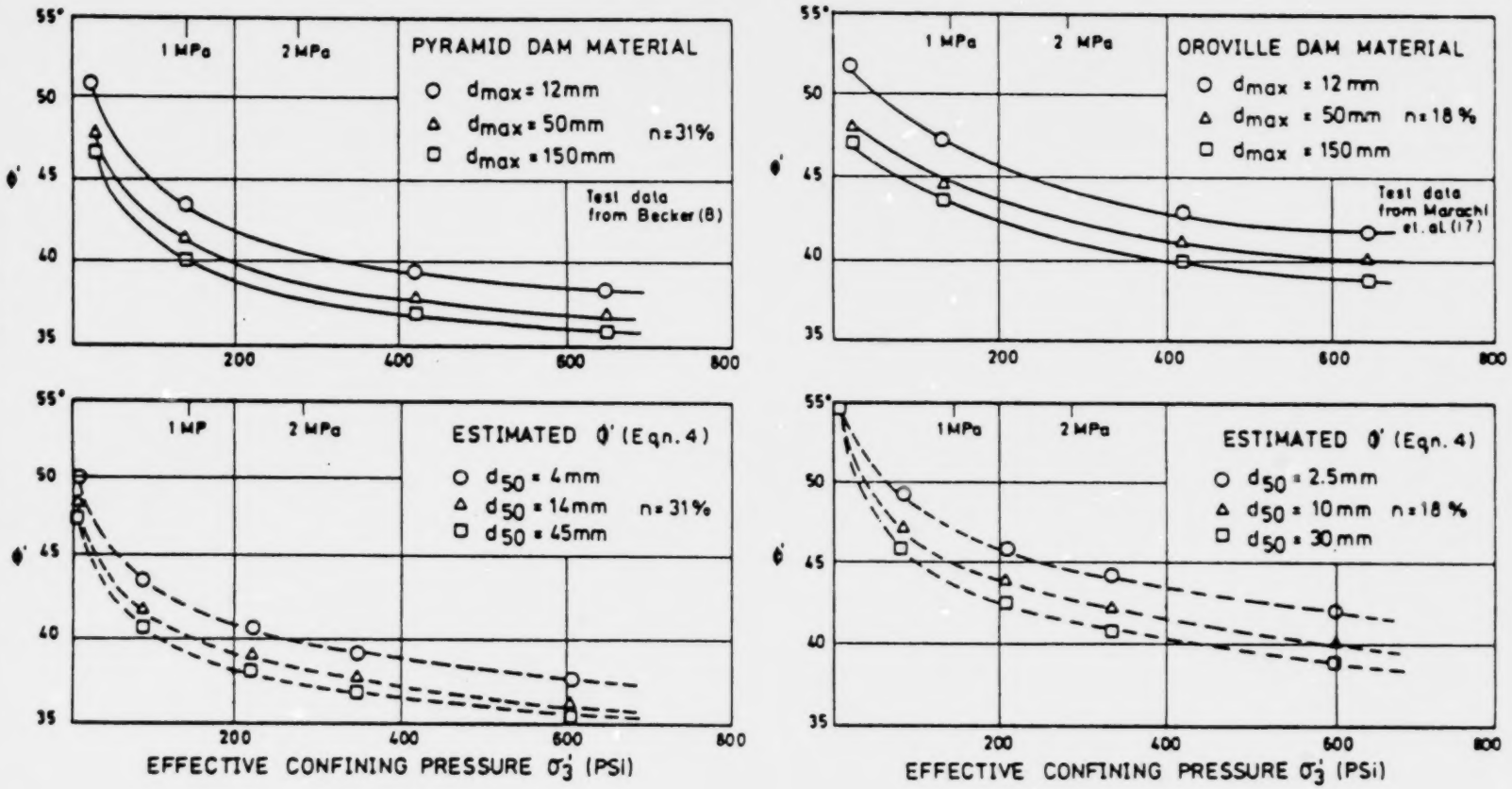


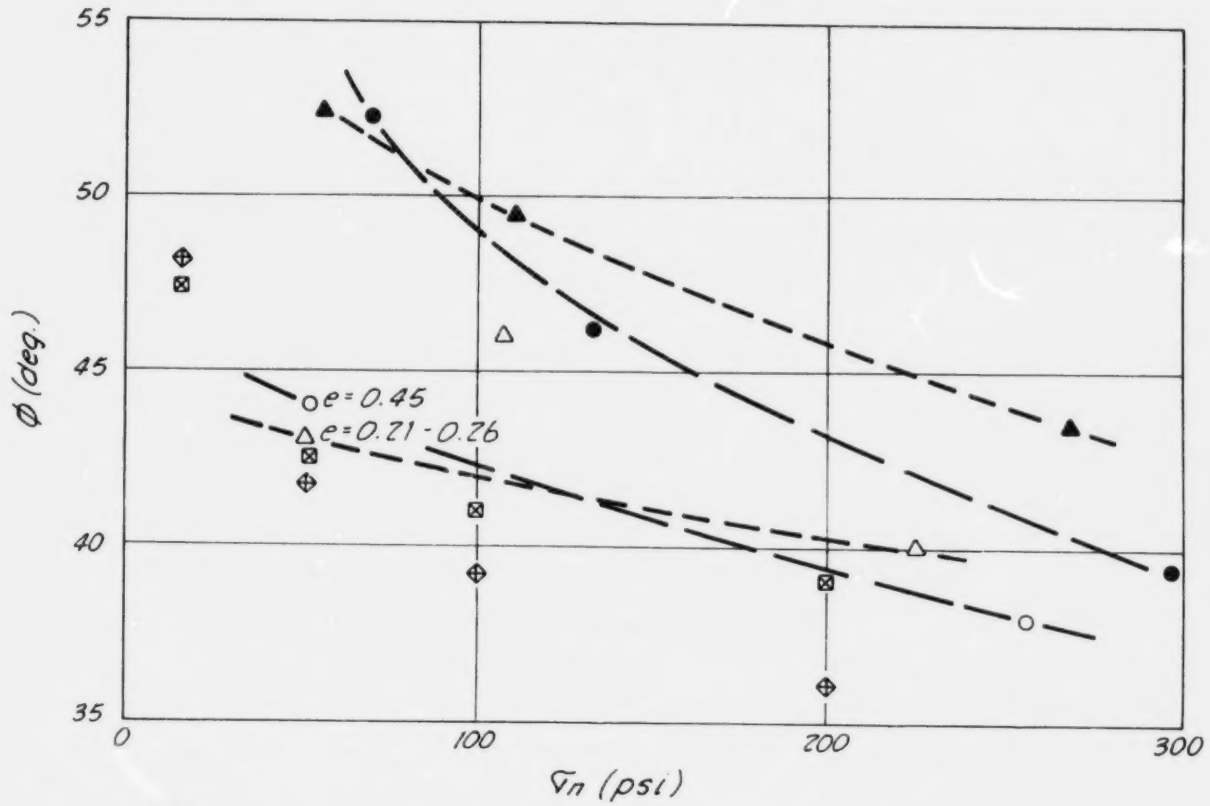

COMPARISON OF TRIAXIAL STRENGTH TEST RESULTS WITH PREDICTION (Barton and Kjaernsli, 1981)

Figure 6.4

Golder Associates

COMPARISON OF LARGE SCALE DIRECT SHEAR TEST RESULTS WITH SMALLER SCALE TESTS AND WITH PREDICTION

Figure **6.5**

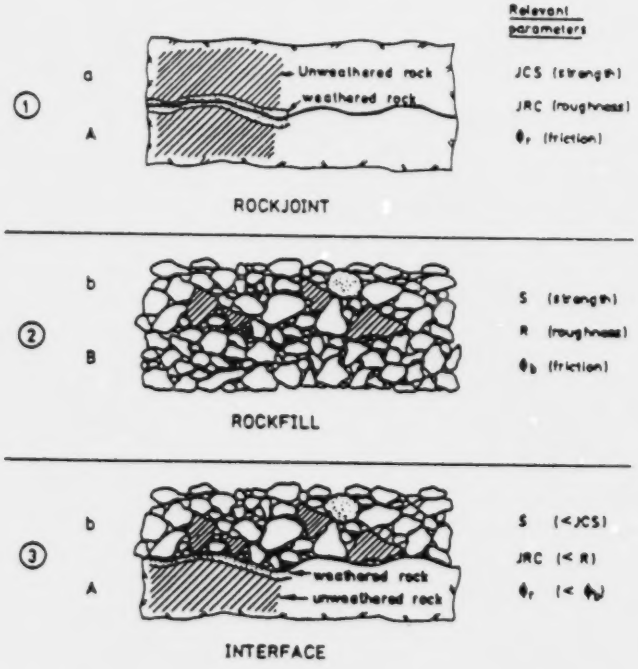



LEGEND:

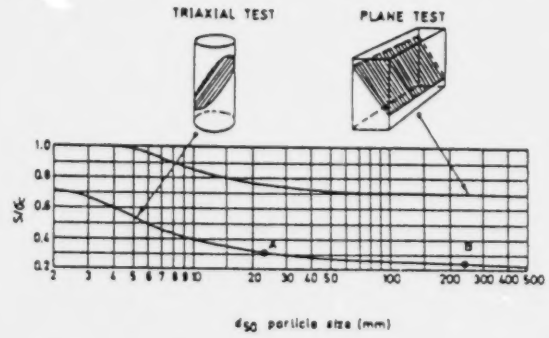
| | | |
|---|---|---|
| △ | Durable Sandstone | Large direct |
| ○ | Non-durable Shale | shear 1.5m × 1.5m |
| ▲ | Durable Sandstone | Small direct |
| ● | Non-durable Shale | shear 0.15m × 0.15m |
| ⊕ | Barton & Kjaernsli prediction | |
| ⊠ | " " " | |

(from Hribar, Dougherty, Ventura & Yavorsky)

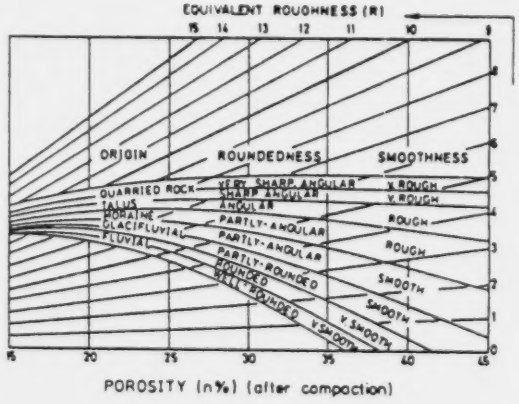
PROJECT NO. 862-1231E DRAWN REVIEWED DATE June '87

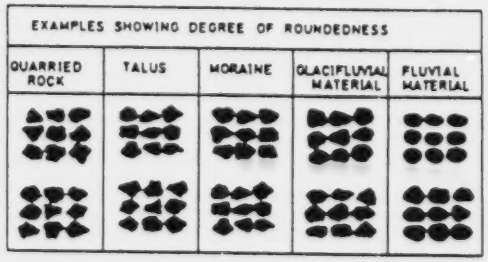



Empirical Approach to Shear Strength Estimation for Rock Joints, Rockfill, and Rock Interfaces




—Method of Estimating Equivalent Strength ($S$) of Rockfill, Based on Uniaxial Compression Strength ($\sigma_c$), and on $d_{50}$ Particle Size




—Method of Estimating Equivalent Roughness ($R$) Based on Porosity of Rockfill, Origin of Materials, and Degree of Roundedness and Smoothness of Particles

SUMMARY OF SHEAR STRENGTH PREDICTION METHOD PROPOSED BY BARTON AND KJAERNSLI (1981)

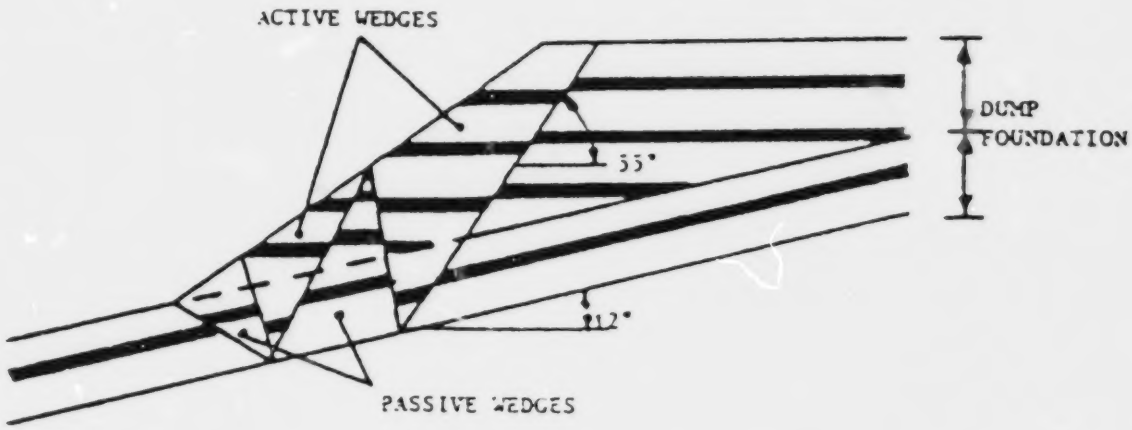
Figure 6.6

Golder Associates

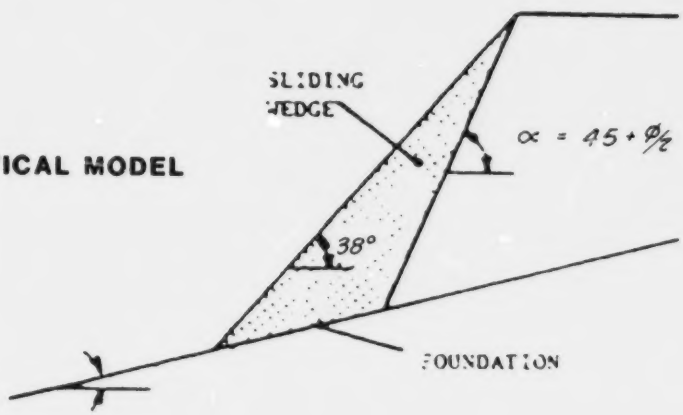
SCALE MODEL AND ANALYTICAL MODEL FOR GRANULAR FILL ON WEAK FOUNDATION (Blight)

Figure **6.7**

MODEL OF ROCK-DUMP ON INCLINED FRICTIONAL FOUNDATION



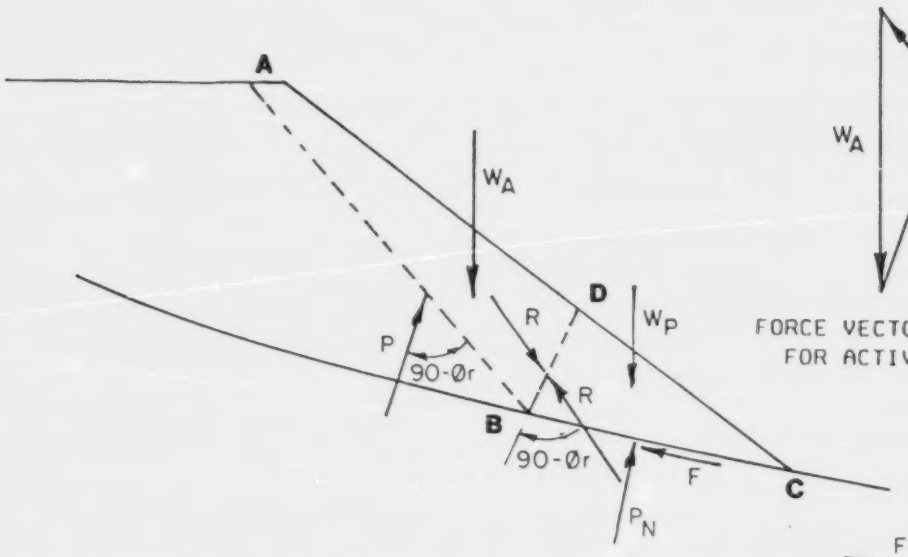

**ANALYTICAL MODEL**




PROJECT NO. 862-1731E DRAWN – REVIEWED DATE June '87

GENERAL WEDGE MECHANISM (Campbell, 1986)

Figure **6.8**



FORCE VECTOR DIAGRAM FOR ACTIVE WEDGE

$W_A$ = Effective weight of active wedge

P&R = Resultant forces on boundaries of active wedge assuming friction angle for waste rock fully mobilized

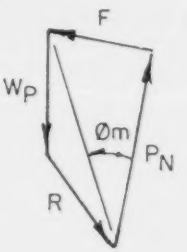
$W_T$ = Effective weight of toe wedge

$P_N$ = Effective force acting normal to base of toe wedge

F = Mobilized base friction for limiting equilibrium of toe wedge

$Ø_r$ = Angle of internal friction for waste rock

$Ø_m$ = Mobilized friction angle on base of toe wedge



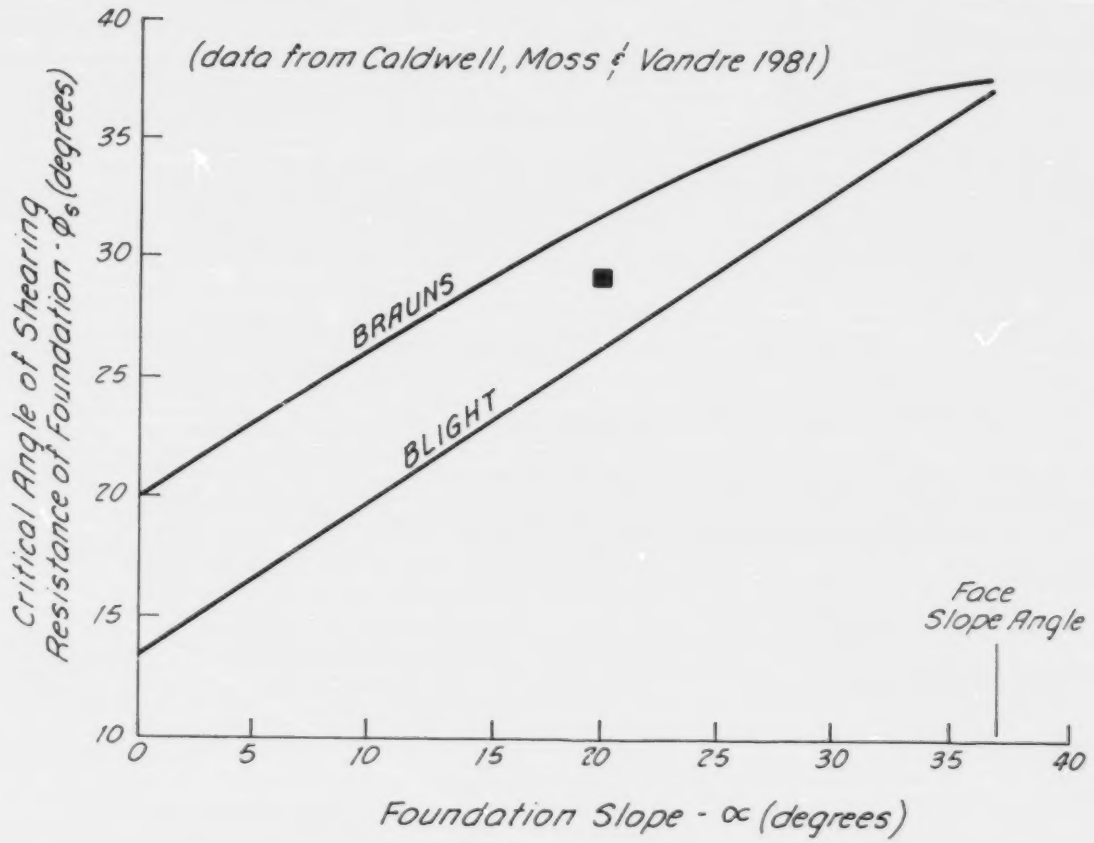

FORCE VECTOR DIAGRAM FOR TOE WEDGE

An illustration of the forces that act on the active wedge and on the toe wedge, and the double wedge method for assessing the stablity of the face of a dump. Along the inclined boundaries of the active wedge, the shear strength parameters for the waste rock are assumed to be fully mobilized. Limiting equilibrium is assumed, and the force 'R' that acts across the mutual boundary between the active and the toe wedges is caculated. The force 'R' is then applied to the toe wedge, and the mobilized friction angle on the base of the toe wedge corresponding to limiting equilibrium is caculated. An indication of the factor of safety for the toe wedge is provided by the ratio of the tangent of the base friction angle to the tangent of the mobilized base friction angle.

PROJECT NO. 862-1231E DRAWN - REVIEWED DATE June '87

**CRITICAL ANGLE OF SHEARING RESISTANCE FOR SLOPED FOUNDATIONS**

Figure **6.9**




■ - Typical result for method shown on Fig. 6.10 (Campbell)

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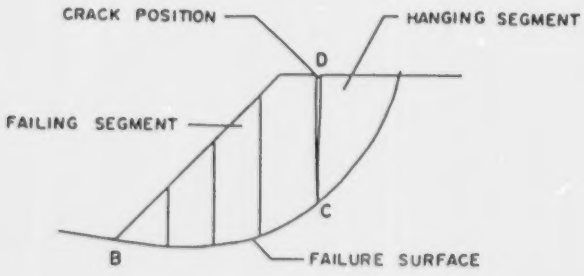
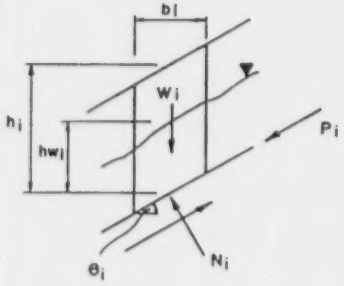



$W_i = b_i h_i \gamma$

$N_i = \sigma_T - U_i$
$= W_i \cos\theta - U_i$

$S_i = b_i c / \cos\theta$

$DF_i = W_i \sin\theta + a W_i \cos\theta$

$RF_i = S_i + N_i \tan\phi$

$P_i = RF_i - DF_i + P_{i-1}$

c = cohesion on base

$\phi$ = soil friction

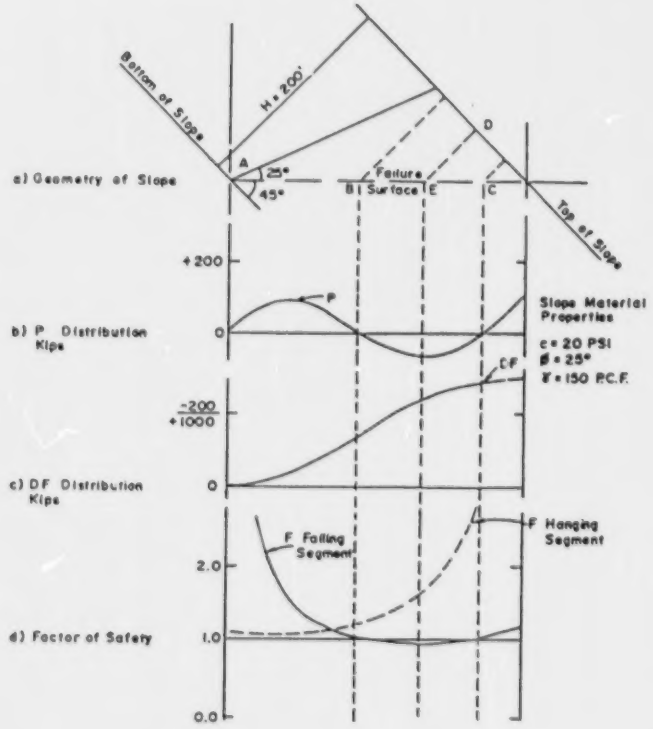
a = earthquake acceleration

U = water pressure

The factor of safety of the segment below the right hand side of slice is thus

$$F = \frac{\sum_{i} P_i}{\sum_{i} (DF)}$$

SOLUTION OF Y=O METHOD




DISTRIBUTION OF P, DF, F FAILING SEGMENT & F HANGING SEGMENT ALONG FAILURE SURFACE

FROM ROBERTSON (1977)

Y = O METHOD

Figure 6.10

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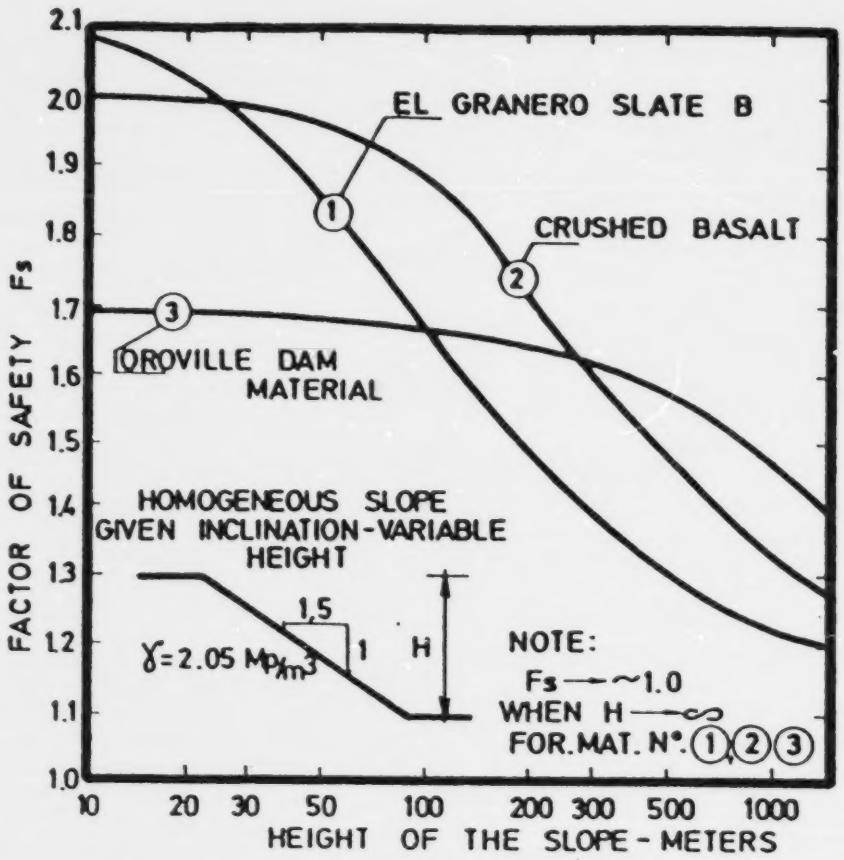
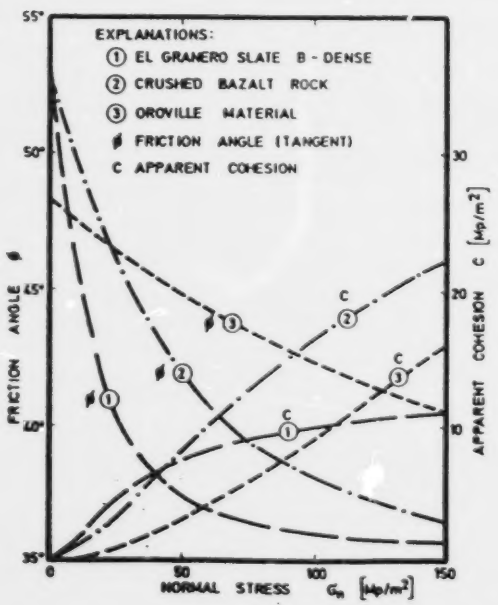
EFFECT OF SLOPE HEIGHT ON FACTOR OF SAFETY WITH STRESS-DEPENDENT FRICTION ANGLE

Figure **6.11**




from Maksimovic (1979)

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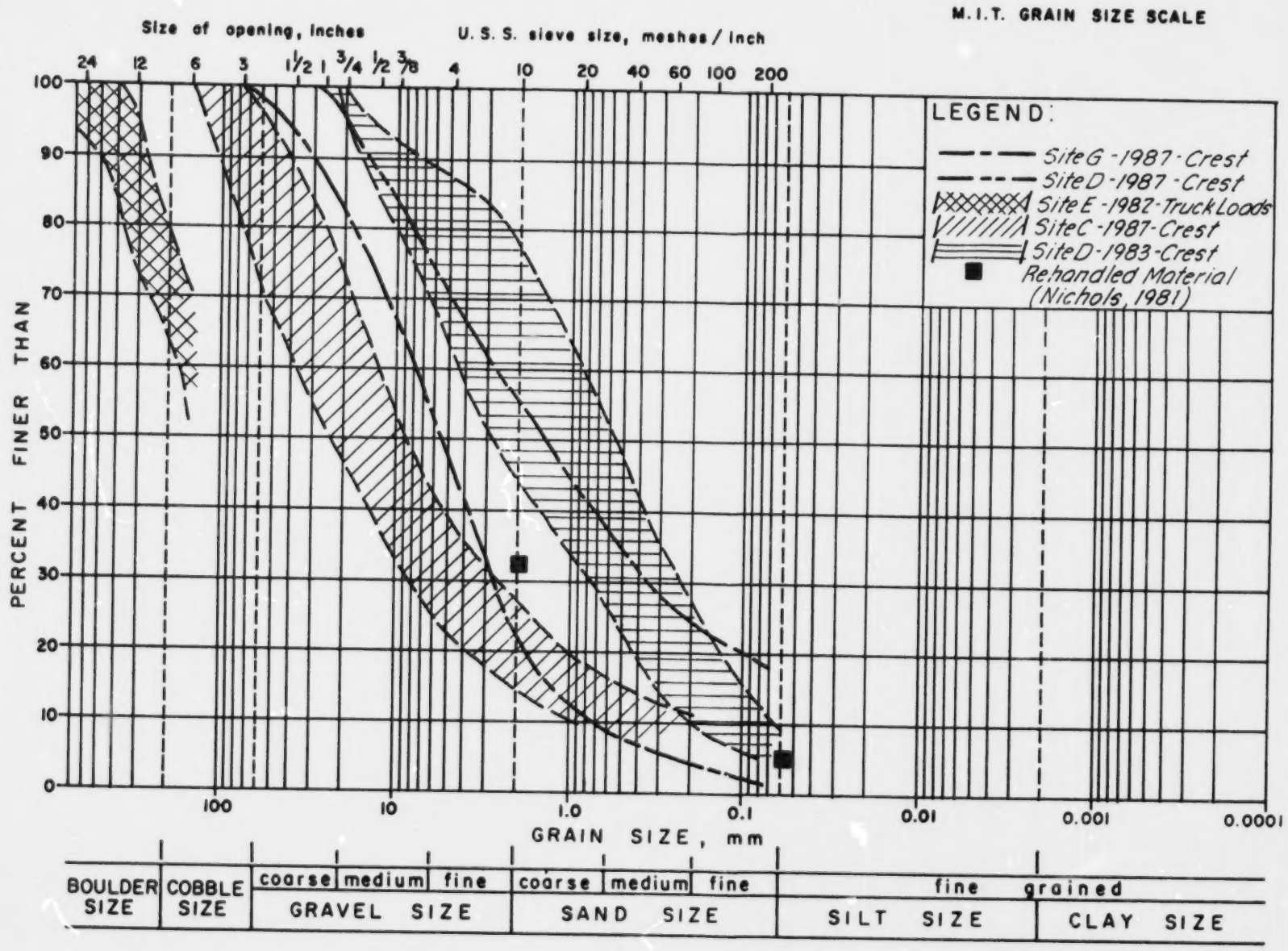
PROJECT NO. 862-1231E DRAWN - REVIEWED DATE June '87


M.I.T. GRAIN SIZE SCALE
Size of opening, inches
U. S. S. sieve size, meshes / inch
24 12 6 3 1½ 1 ¾ ½ ⅜ 4 10 20 40 60 100 200
PERCENT FINER THAN
100 90 80 70 60 50 40 30 20 10 0
LEGEND:
Site G - 1987 - Crest
Site D - 1987 - Crest
Site E - 1982 - Truck Loads
Site C - 1987 - Crest
Site D - 1983 - Crest
Rehandled Material (Nichols, 1981)
100 10 1.0 0.1 0.01 0.001 0.0001
GRAIN SIZE, mm
BOULDER SIZE
COBBLE SIZE
coarse medium fine
GRAVEL SIZE
coarse medium fine
SAND SIZE
fine grained
SILT SIZE
CLAY SIZE

GRAIN SIZE DISTRIBUTION

WASTE DUMP SAMPLES

Figure 7.1

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PARTICLE SIZE VARIATION WITH POSITION ON DUMP FACE

Figure **7.2**

120 m ABOVE TOE

160 m ABOVE TOE

180 m ABOVE TOE

TOE OF DRAIN

40 m ABOVE TOE

80 m ABOVE TOE

CENTRE OF DRAIN

0 1 m

Scale of Particle Size, m

A series of photographs at selected intervals of height above the toe of Lawyers Nest Resources' West Line Creek waste rock dump. The photos illustrate the reduction in the size of the rock fragments proceeding from the dump toe toward the dump crest. All photographs were taken looking normal to the surface of the waste rock, with the camera at a distance 3.0 meters from the surface. The sizes of individual rock fragments are indicated by the scale.

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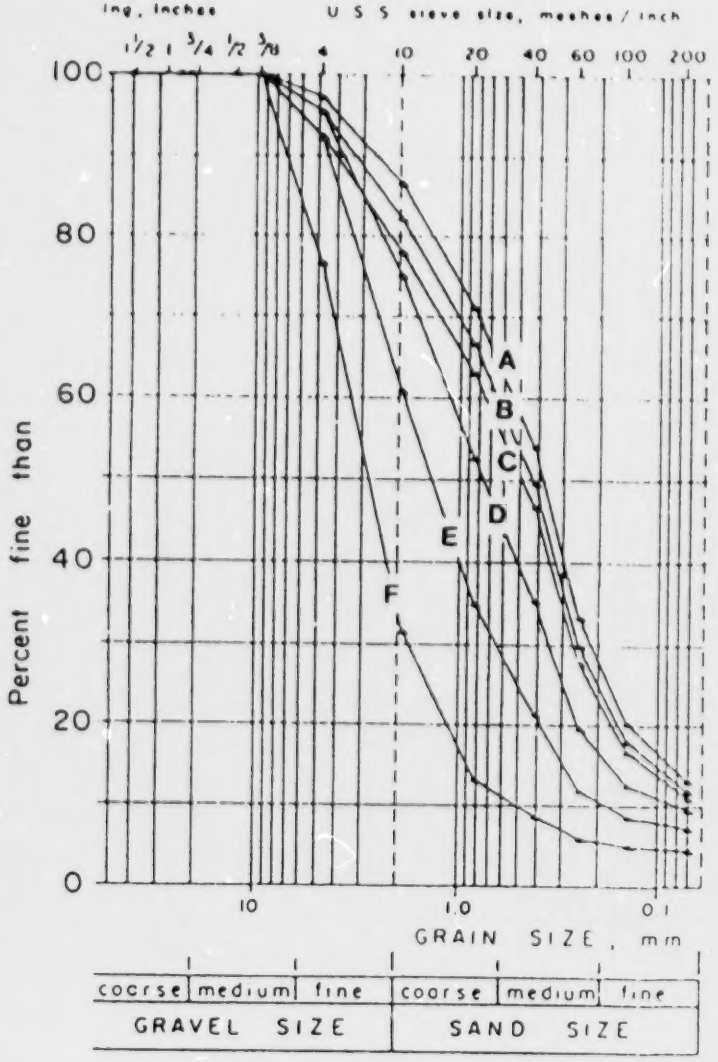
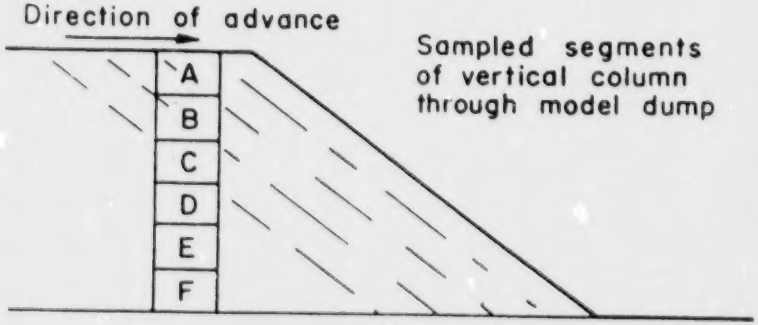
PROJECT NO. 862-1231E DRAWN - REVIEWED DATE June '87




Grain size curves showing particle size distribution of material within a vertical column through a model dump, and illustrating that downward migration of particles from the upper to the lower regions of the dump cannot occur.

RESULTS OF SCALE MODEL SIMULATION OF WASTE ROCK MATERIAL SEGREGATION

Figure 7.3

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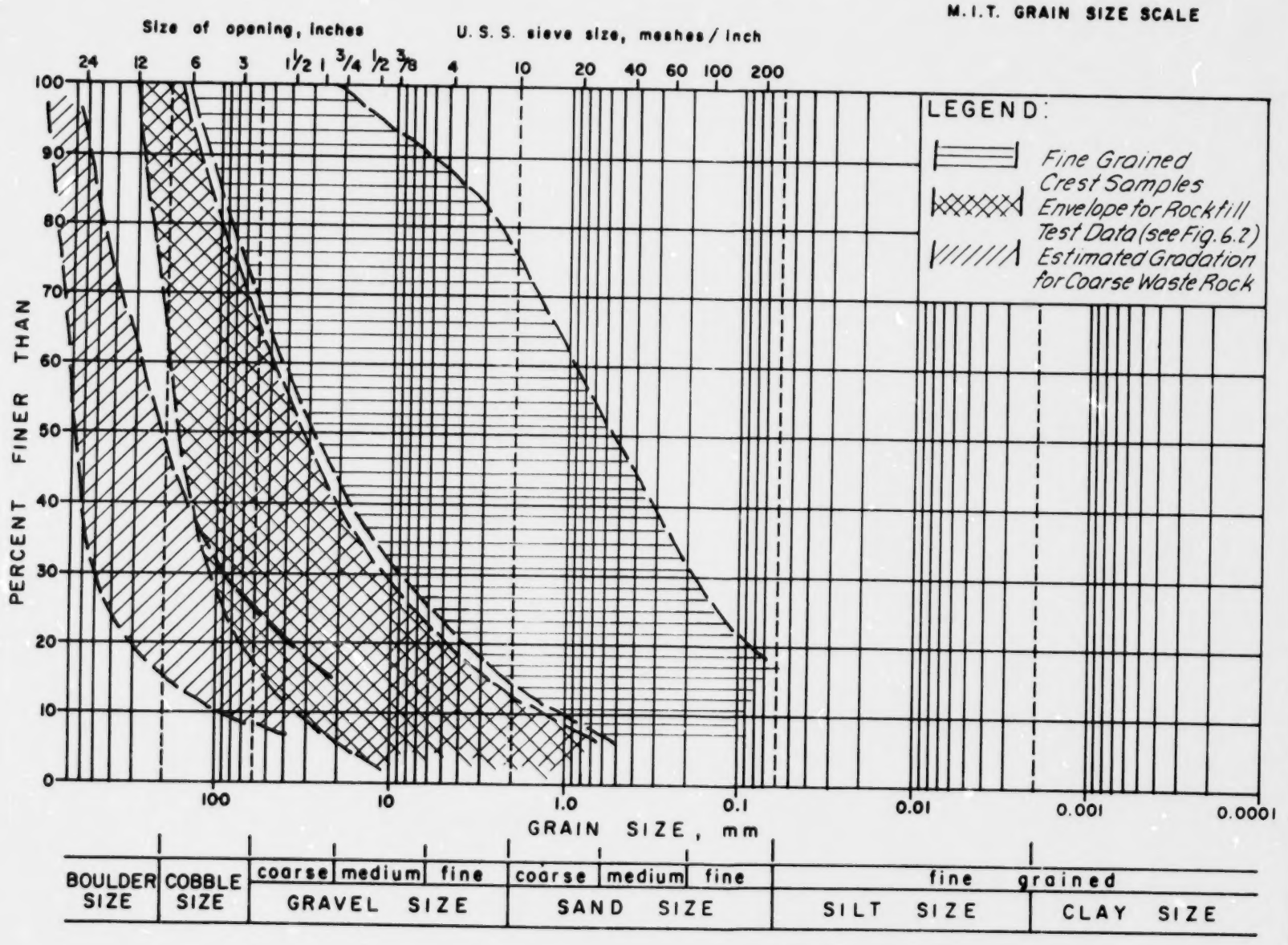
PROJECT NO. 862-1231E DRAWN - REVIEWED - DATE June '87


M.I.T. GRAIN SIZE SCALE
Size of opening, inches
U. S. S. sieve size, meshes / inch
24 12 6 3 1½ 1 ¾ ½ ⅜ 4 10 20 40 60 100 200
PERCENT FINER THAN
100 90 80 70 60 50 40 30 20 10 0
LEGEND:
Fine Grained
Crest Samples
Envelope for Rockfill
Test Data (see Fig. 6.2)
Estimated Gradation
for Coarse Waste Rock
100 10 1.0 0.1 0.01 0.001 0.0001
GRAIN SIZE, mm
BOULDER SIZE
COBBLE SIZE
coarse medium fine
GRAVEL SIZE
coarse medium fine
SAND SIZE
fine grained
SILT SIZE
CLAY SIZE

GRAIN SIZE DISTRIBUTION

RANGE FOR WASTE ROCK AND AVAILABLE TESTS

Figure 7.4

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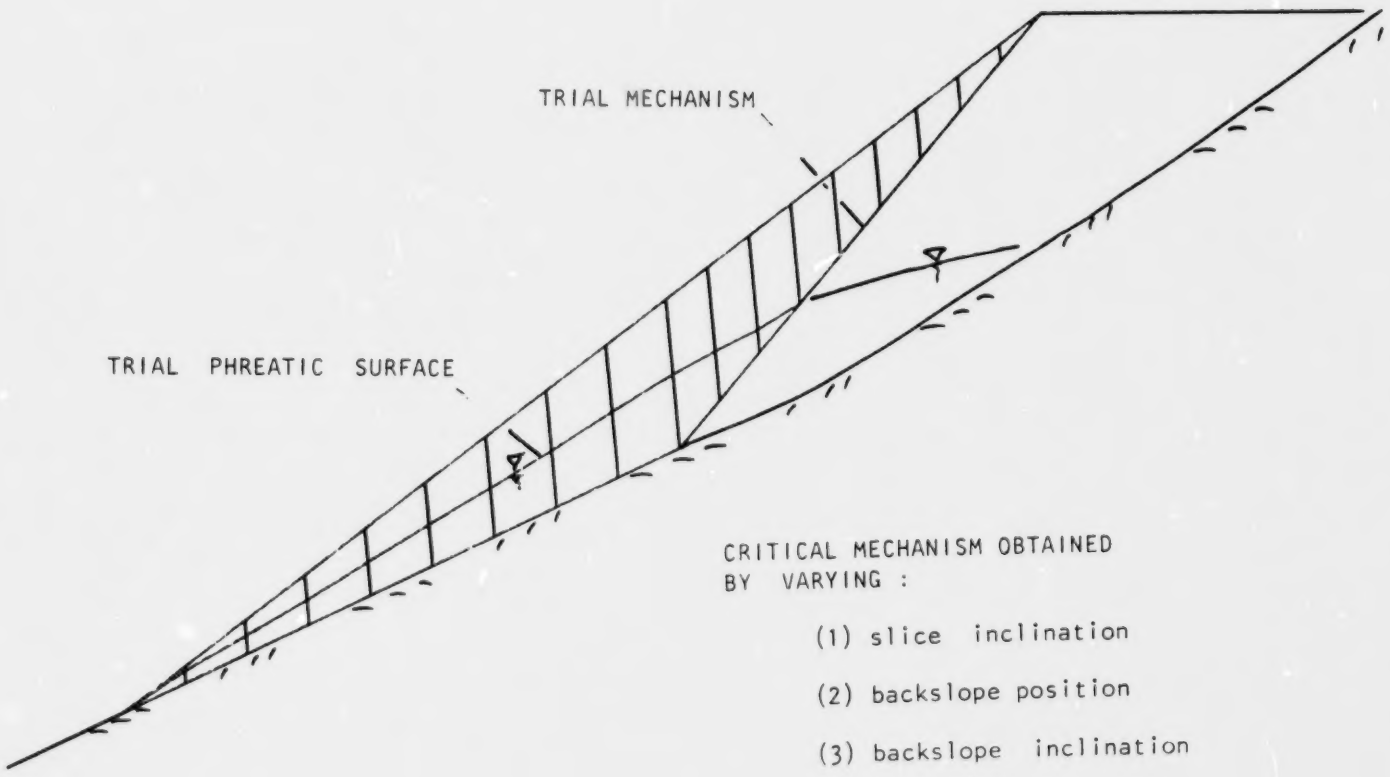
PROJECT NO. ..... DRAWN ..... REVIEWED ..... DATE .....




CRITICAL MECHANISM OBTAINED BY VARYING :

(1) slice inclination

(2) backslope position

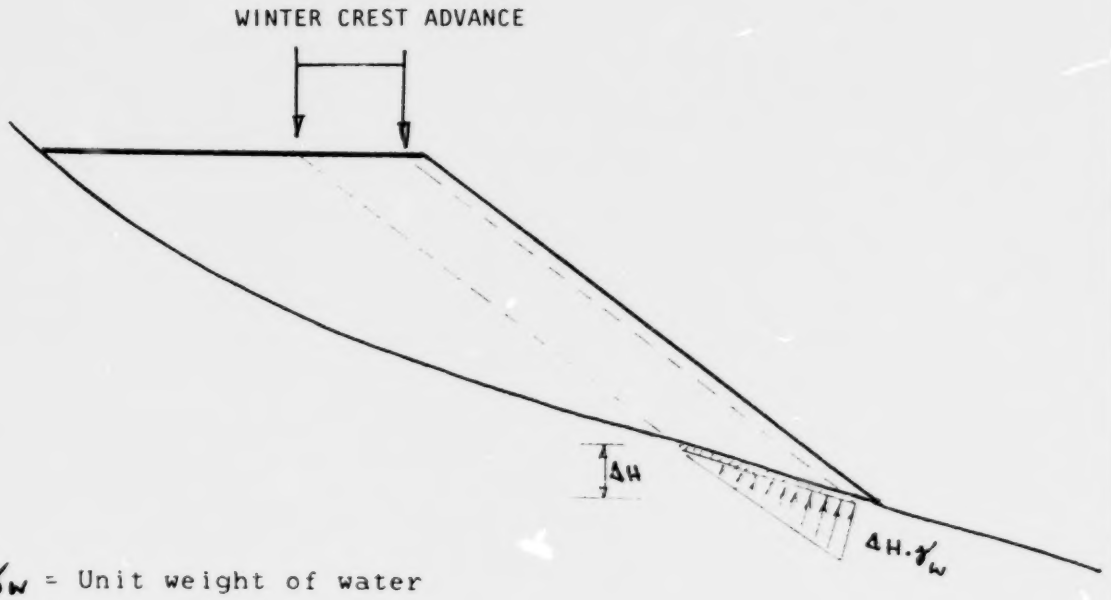
(3) backslope inclination

NON-VERTICAL SLICE MECHANISM FOR STABILITY ANALYSIS

Figure 7.5

Golder Associates

| FAILURE MECHANISM OF FROZEN FOUNDATION WITH HYDROSTATIC PRESSURE | Figure 7.6 |
|---|---|



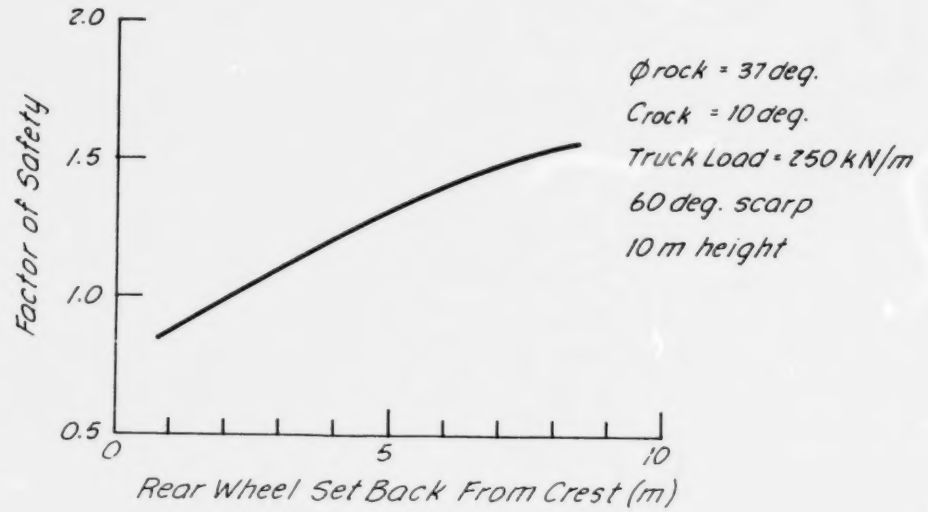

$\gamma_w$ = Unit weight of water

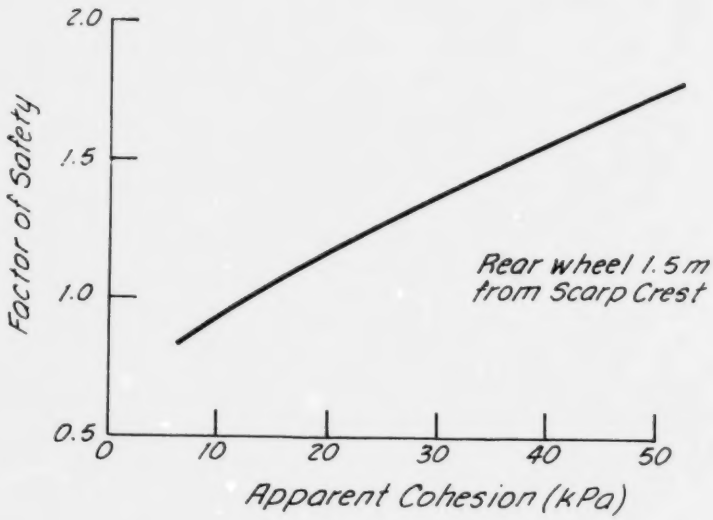
$\Delta$ H = Difference in elevation between upper lower surfaces of frozen crust

Build-up of hydrostatic pressure beneath a frozen crust buried by advance of spoil during winter reduced shearing resistance within foundation soils at base of frozen crust, permitting toe to move downslope. This movement at the toe removed support for materials within the upper regions of the dump face which also became involved in the failure.

Not to scale)

| SAFETY OF DUMP TRUCK NEAR FAILURE SCARP | Figure 7.7 |
|---|---|




PROJECT NO. 862-1231 E DRAWN BAD REVIEWED DATE June '87

# APPENDIX A

PHOTOGRAPHS

PHOTOGRAPH ONE

Melting of snow over approximately the middle third of the dump face is clearly evident. This phenomenon may be due to convective flow of warm air through more pervious zones above the resisting toe wedge and below finer waste inthe upper part of the dump. At other locations vapour has been observed to erupt from localised areas immediately above wrap-around dumps. Ambient conditions of temperature and humidity are believed to change very gradually within the overall mass of waste dumps.

PHOTOGRAPH TWO

A pattern of horizontal striations is clearly evident over the upper one third approximately of the dump face. These striations are considered to represent the upper limits of parallel 'planes' of shear within the 'Active' or driving wedge in the upper part of the dump. Discrete lines are generated as a result of variations in the texture of the surface.

PHOTOGRAPH THREE

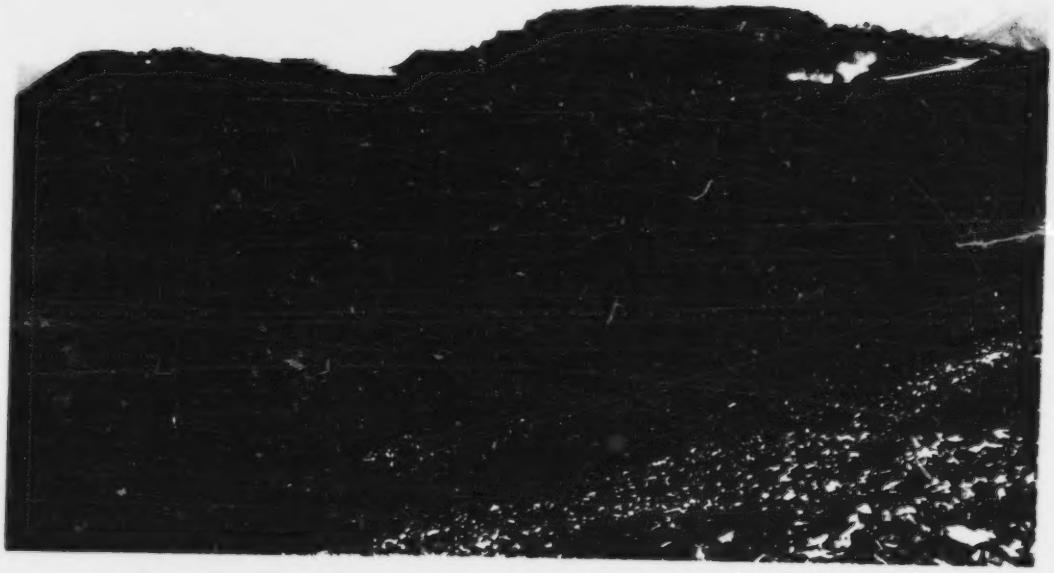
The distant dump face profile illustrates that the slope is not uniform. The lower quarter and the upper half to one third of the face is relatively steep. In this case the total variation in face angle is estimated to be about 3 degrees. Slopes have been observed which are inclined at angles of up to 42 degrees both in the crest and toe regions.

PHOTOGRAPH FOUR

This photograph illustrtates consistent depopsition of large durable particles at the base of dumps with large face heights. Figure 7.2 shows the photographic record of a traverse down the face of this dump. Overall it is clear that the average particle size of material increases with decreasing elevation.

PHOTOGRAPH FIVE

The initial stages of a dump founded on steep terrain inclined at more than 25 degrees. A stable platform was advanced along the axis of a prominent v-shaped gully, as illustrated in figure 4.5. Three-dimensional wedging action within the gully is estimated to have increased the indicated factor of safety against base sliding by 10 to 15 per cent.

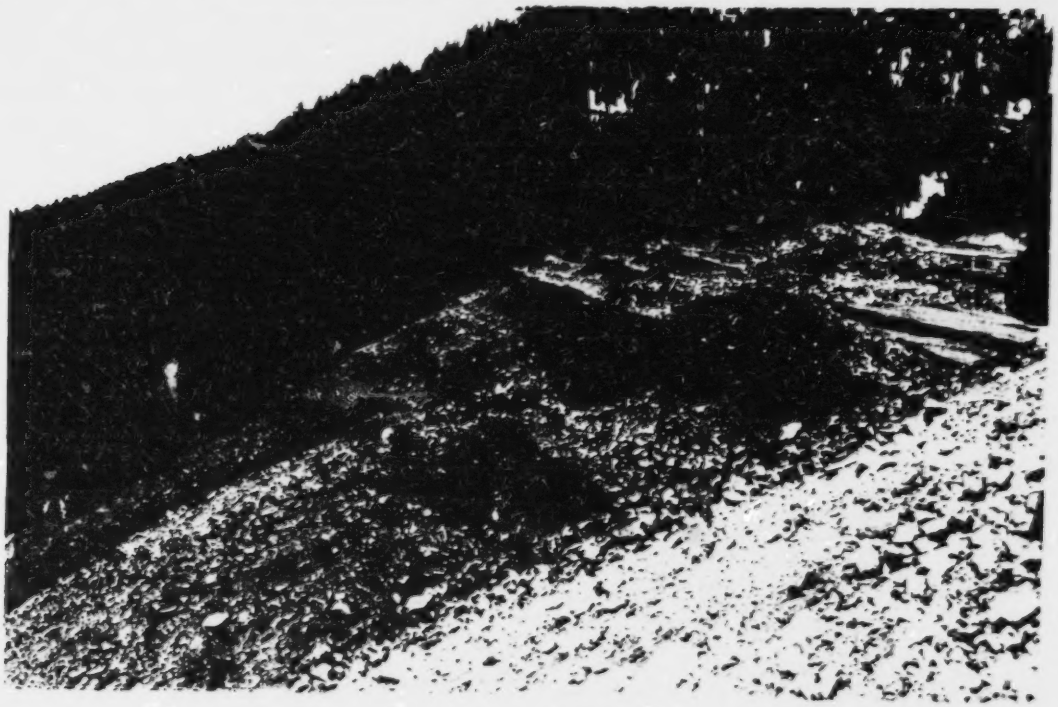


PHOTOGRAPH SIX

Relatively fine-grained waste rock from highly disturbed and weathered strata have been consistently consigned to this dump. Several large failures have occurred. Several of these events have occurred relatively suddenly compared to other dump failures. It is believed that ground water discharge may be a factor compounded by poor drainage within the dump. Note the absence at the toe of a skirt of particularly coarse material normally visible at other sites.

PHOTOGRAPH SEVEN

In the middle distance a segment of crest 10 to 20 metres wide has slumped virtually intact by a similar vertical distance. A number of steep scarps are visible signifying that an significant amount of apparent cohesion is present. Such scarps have been observed with heights of 10 to 15 metres particularly in the winter months. As can be seen in the foreground, gradual ravelling of the scarps flattens the slope to the angle of repose. In the short-term dumping operations can be severely curtailed by extensive steep scarps.

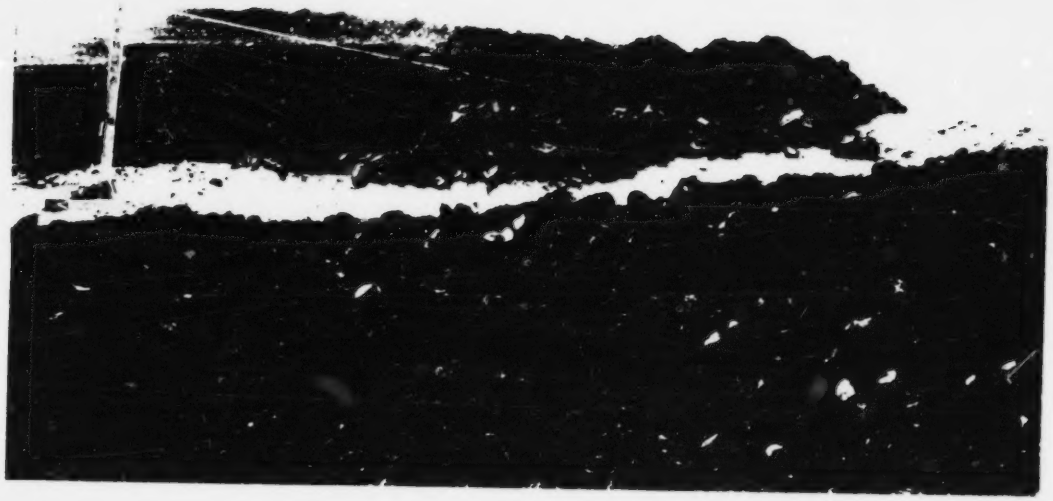


PHOTOGRAPH EIGHT

A series of scarps may often occur similar to the surveyed profile shown on figure 5.41. This type of deformation is considered to be similar to that shown on photograph Two.

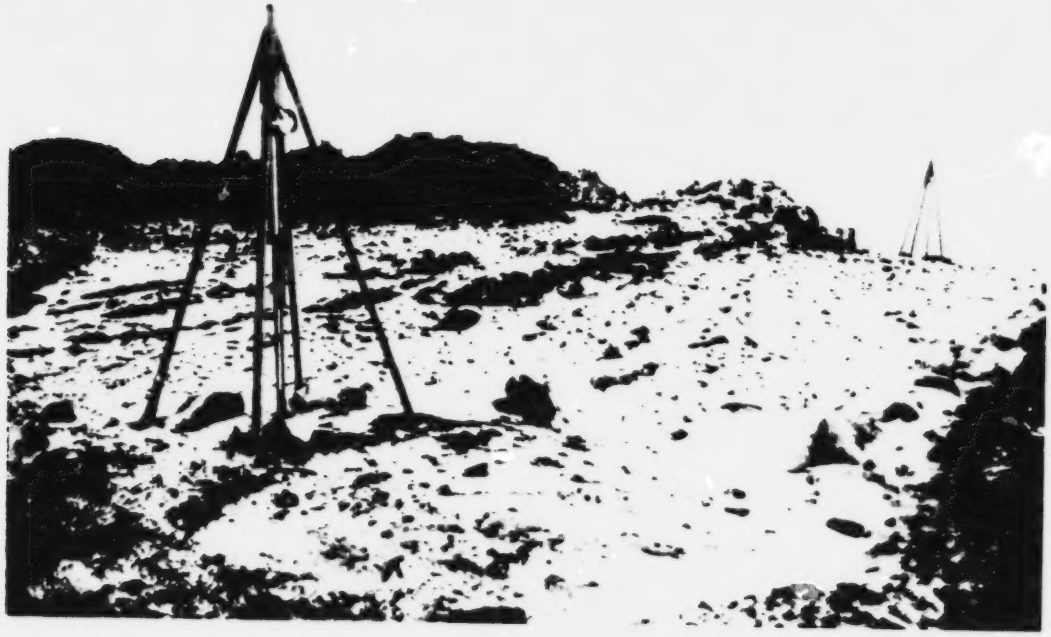
PHOTOGRAPH NINE

The surface of a dump platform exhibits increased strength due to the compact state of the material and some capillary tension. With typical crest displacement rates of up to 1 metre per day significant volumes of compacted material are transported well down into the dump.



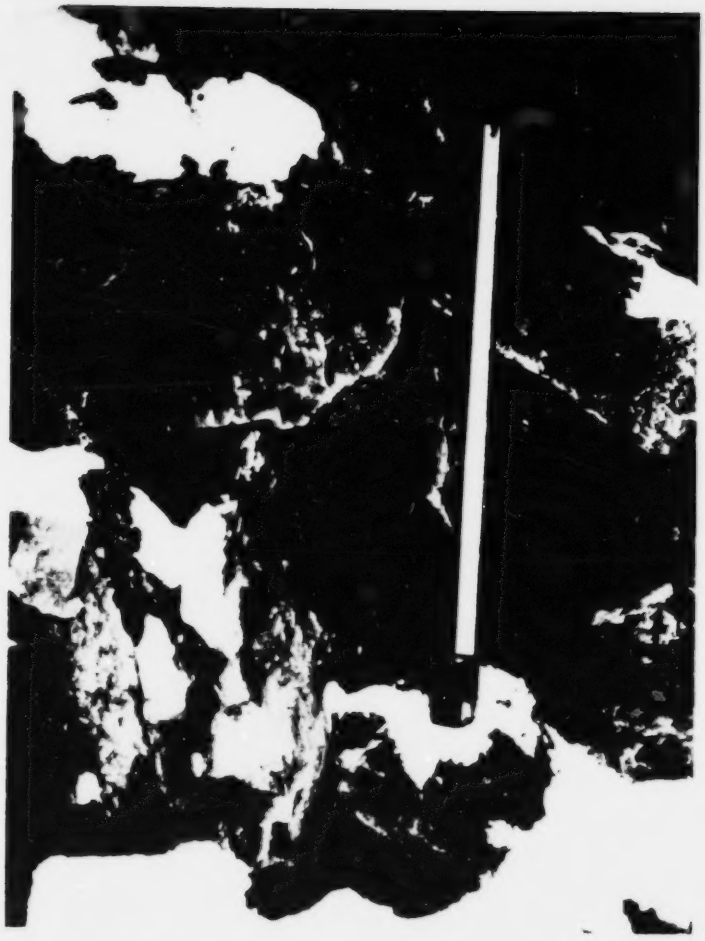
PHOTOGRAPH TEN

A single stand wireline extensometer attached to a peg at the dump crest. The corridor undisturbed by dumping also provides a good opportunity for visual observation of the development of cracks and steps.



PHOTOGRAPH ELEVEN

A two stand wire-line extensometer with the outer stand on settling segment of crest. The wire may be attached to a peg located either some distance down the face or, if large deformations are occurring, to a potentially unstable segment of crest. These devices are robust and simple to operate.



PHOTOGRAPH TWELVE

This photograph shows waste rock at the base of the No. 1 spoil at the Fording property. The rock shown in this photograph is at an estimated height of 1.5 m above the surface of contact between the base of the spoil and the natural foundation, and approximately 50 m below the original surface of the No. 1 spoil. This exposure was examined to make an assessment of the degree of fracturing that has occurred as a result of high point-to-point contact stresses due to the weight of the overlying waste rock fill. The materials shown in this photograph represent rock fragments within the zone of coarse rock that results from separation of particle sizes on the dump face, and the accumulation of these large fragments in the region of the dump toe.

The scale as included in this photograph represents a distance of 0.5 m and the scale is marked in 1 decimeter increments.

Note the rock-to-rock contact at a level approximately 2.9 decimeters below the top, and a distance approximately 2 decimeters to the left of the scale. The upper edge of the rock at the base of the sandstone block exhibits a series of vertical cracks that extend a distance of approximately 3 cm below the surface of contact. This pattern of cracking is undoubtedly the result of high contact stresses imposed by the overlying block of sandstone. The left-hand edge of the sandstone block is also fractured, but to a lesser degree than is the upper surface of the underlying block. For the most part, the small fragments produced by this fracturing remain in place.

To the left of the case of the measuring tape, note the two quasi vertical fractures that extend completely through this block of rock. This fracturing appears to have been caused by the stresses imposed by the weight of the overlying rock. If these fractures had been extant at or before the time that the rock arrived in this position, the block would have separated in the course of transit down the face, and the separate pieces would have come to rest at separate locations.

Immediately to the right of the scale, in the region between the four decimeter and five decimeter marks, note the series of quasi vertical fractures in the wedge-shaped zone above and to the right of the small patch of snow upon which the case of the tape rests. This fracturing has also been produced by high contact stresses imposed by the overlying block.

# APPENDIX B

# DATA BASE

## DATABASE LEGEND

DUMP CODE - Same letter for same property
- Same number for same dump

PROFILE TYPE - Refer to figure 4.3 for for foundation surface profile type

3-D EFFECT - YES indicates that 3-D wedge action a possible factor

FOUNDATION MATERIAL - C = Colluvium S = Spring discharge
T = Glacial Till O = organics
F = Frozen

GROUND WATER - YES indicates that it is a possible factor

ACTIVE - A = Active I = Inactive with respect to dumping

DUMPING RATE in Bank Cubic Metres per utilised metres of crest per day

RQ - Waste Rock Quality : P=Poor, M=Moderate G= Good

? W - Y indicates that weather conditions believed to be a factor

MONitor Type : WL2 is wireline with 2 stands
SRVY is measurement of one/more components of displacement
TVS is measurement of all components of displt

PEAK RATE is last value recorded prior to failure

dT is time preceding failure of peak rate

dT PAST is time preceding failure of geven rate of crest displt

B.G. is average background rate prior to failure event

VOL = Volume of slide mass as estimated by mine personnel

Width = width of failure scarp at crest

RUNOUT = Runout distance of slide debris from crest

| EVE NT: NO | DUMP CODE | EVENT DATE | GEOMETRY TOE SL-OPE (M) | HEI-GHT | ILE: TYPE | 3-D: EFF: ECT | MAT'L: ? TYPE | ? GRND WTR | RATE ACT: IVE | RQ: BCM/M: /DAY | W?: | TEMP: DEG | PPT (MM) | FOUNDAT'N CONDIT'NS DUMPING MON. | FREQ. TYPE | PEAK DISPL RATE (M/ DAY) | dT TO: FAIL. (HRS) | dT PAST 0.5M/: DAY | dT PAST 1 M/D: | dT PAST 2 M/D: | B.G. M/DY: | VOL. LCM | WID- TH | RUN OUT (M) | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| MAJOR/RAPID/PREDICTED FAILURE OF ACTIVE DUMPS | | | | | | | | | | | | | | | | | | | | | | | | |
| 1 | C | 1 :06/11/83 | 32 | 235 | A | Y | CT | N | A | | | Y | 11. | HEAV | WL2 :4 HR | 2.8 | | | 21 | 10 | :0.5 | 110 | 65 | 900 | NEAR 10 DAYS OF 0.9M/DAY DISPLACEMENT |
| 3 | C | 3 :09/21/84 | 26 | 225 | A | Y | CT | N | A | | | Y | 7.2 | HEAV | WL2 :2 HR | 4 | | | 92 | <6 | :0.7 | 500 | 170 | 900 | |
| 5 | C | 4 :07/24/84 | 26 | 210 | A | Y | CT | Y | A | | | N | 20 | 14 | WL2 :2 HR | 1.7 | | | 290 | | | 300 | 100 | 900 | GW SEEN FLOWING FROM TOE A FEW DAYS EARLIER |
| 6 | C | 2 :08/16/85 | 28 | 195 | A | N | CT | N | A | | P | Y | | 50 | WL2 :2 HR | 15.5 | | | >40 | | | 140 | | 800 | |
| 11 | C | 5 :09/16/85 | 28 | 300 | A | N | CT | N | A | | P | Y | | 37 | WL2 :2 HR | 2 | | | 120 | | :0.8 | 225 | | 500 | |
| 13 | C | 15 :11/23/86 | 28 | 290 | A | Y | CT | N | A | | | S | >0 | | WL2 :2 HR | | | | 300 | | | 45 | | 500 | |
| 15 | C | 8 :11/16/85 | 27 | 300 | A | Y | CR | | I | | | N | <0 | 37 | WL2 :4 HR | 0.7 | | 80 | <34 | <34 | :0.3 | 70 | 120 | | DELAYED FAILURE. |
| 16 | C | 9 :06/17/85 | 36 | 140 | B | Y? | C? | | I | | | N | 10 | 21 | - | - | - | | | | | 55 | 90 | 470 | DELAYED FAILURE. |
| 17 | C | 10 :08/15/80 | 15 | 47 | A | N | O? | | A | | G | Y | 14 | HEAV | - | - | - | | | | | 75 | 220 | | MUDFLOW ? |
| 19 | C | 6 :05/05/72 | 29 | 130 | A | | CT | | A | | | S | | | - | - | - | | | | | 50 | | 500 | VERY HEAVY SNOWFALL THIS YEAR. BUT GNDW NOT FELT IMPORTANT. |
| 20 | C | 6 :05/27/72 | 29 | 130 | A | | CT | | A | | | S | | | - | - | - | | | | | 160 | | 650 | VERY HEAVY SNOWFALL THIS YEAR. BUT GNDW NOT FELT IMPORTANT. |
| 24 | C | 6 :11/08/74 | 29 | 110 | BA | N? | O | | A | | | N | 0 | 50 | WL2 :1 DY | 3.7 | | 96 | 48 | | :0.3 | | 160 | 500 | |
| 36 | E | 2 :06/29/82 | 25 | 150 | AB | | CT | | A | | | Y | 20 | 40 | SRVY: 1 DY | 1.1 | | 100 | 34 | <24 | 0 | 650 | 200 | 1200 | SPATIAL VARIATIONS MEASURED. |
| 38 | E | 5 :02/08/76 | 30 | 200 | B? | | | | A | | | | 3.2 | 69 | - | - | - | | | | | | | 610 | |
| 40 | E | 7 :11/24/68 | 30 | 70 | A | | T | | A | | | Y | | HEAV | - | - | - | | | | | 150 | 100 | | HEAVY RAIN ONTO SNOW PACK |
| 44 | E | 5 :05/25/73 | 35 | 120 | C? | Y | | | A | | | S | 16 | MELT | - | - | - | | | | | 110 | 160 | 2500 | |
| 47 | D | 1 :03/10/86 | 34 | 220 | A | N | C | N | A | | | S? | 8 | MELT | WL2 :2 HR | 13.1 | 24 | 36 | 2 | 2 | :0.1 | | | | MONITOR DATA UNRELIABLE. |
| 48 | A | 1 :06/21/86 | 14 | 200 | AB | N | C | N | A | 53 | M | Y | 20 | 40 | WL1 :2 HR | 5.8 | 96 | 144 | 102 | 30 | :0.5 | 600 | 140 | 300 | FLUCTUATING HI RATES OVER PAST 3 MONTHS |
| 49 | A | 2 :06/30/86 | 16 | 180 | BAB | N | DC | N | A | 74 | 6 | Y | 20 | 40 | WL1 :2 HR | 4 | 120 | 216 | 150 | 50 | :0.7 | 1600 | 200 | 1700 | |
| 54 | A | 3 :09/09/85 | 26 | 220 | ?B | N? | CT | Y | A | 100 | | Y | 10 | 45 | WL1 :2 HR | ? | <100 | 40 | 24 | | :0.7 | | 400 | 1400 | MONITOR AT EDGE OF FAILURE ZONE. LESS THAN 1 DAY WARNING. |
| 62 | B | 6 :07/01/82 | 34 | 100 | AB | Y | | N | A | | | Y | ? | HEAV | WL2 :2 HR | | | | | | | 250 | | 2000 | |
| 69 | C | 2 :05/08/85 | 28 | 190 | A | N | CT | N | A | | | S | 10 | 0 | WL2 :2 HR | 4.3 | | | 60 | 38 | | 50 | | 60 | |
| 70 | C | 5 :08/29/86 | 30 | 215 | A | N | CT | N | A | 150 | | N | | | WL2 :2 HR | | 3 | 65 | >72 | 56 | | | | | DUMP HEIGHT MAY BE HIGHER. |
| 74 | D | 1 :09/07/86 | 30 | 225 | A | N | C | N | A | | | M | 15 | 60 | - | - | - | | | | | | | | MONITOR DATA UNRELIABLE. |
| 75 | D | 4 :03/20/83 | 27 | 160 | A | | S | | A | | P | N | 1.6 | 0 | WL2 :1 DY | 5.5 | | | 86 | 68 | :0.3 | 400 | | 700 | |
| 76 | D | 4 :05/11/83 | 13 | 168 | A | | S | | A | | P | S | 9.3 | 34 | - | - | - | 0.5 | 24 | | | 720 | 250 | 400 | |
| 77 | D | 4 :11/25/85 | 30 | 120 | A | | S | | A | | P | M | 0 | 22R | - | - | - | 1.7 | 120 | 220 | 170 | | :0.3 | | 160 | MONITOR DATA UNRELIABLE./ POOR QUALITY DEBRIS AT TOE |
| 78 | D | 4 :11/11/85 | 30 | 120 | B | | S | | A | | P | M | 0 | 22R | - | - | - | | | | | | :0.3 | | | MONITOR DATA UNRELIABLE./ POOR QUALITY DEBRIS AT TOE |
| 79 | D | 4 :07/12/86 | 18 | 160 | A | | S | | A | | P | N | | 32 | - | - | - | | | | | | :0.3 | 480 | 150 | MONITOR DATA UNRELIABLE./ POOR QUALITY DEBRIS AT TOE |
| 80 | D | 4 :02/12/87 | 35 | 120 | B | | S | | A | | P | | | | - | - | - | 2.8 | <24 | <24 | <24 | | :0.3 | 580 | 200 | MONITOR DATA UNRELIABLE./ POOR QUALITY DEBRIS AT TOE |
| 81 | D | 4 :03/07/87 | 18 | 90 | BA | | S | | A | | P | | | | WL2 :1 DY | 6.3 | 24 | 24 | 24 | | 1 | 285 | 150 | | MONITOR DATA UNRELIABLE./ POOR QUALITY DEBRIS AT TOE |
| 89 | D | 1 :07/01/85 | 30 | 160 | A | Y | C | N | A | | | N | 20 | 2 | - | - | - | | | | | | | | |
| 112 | A | 8 :08/17/84 | 20 | 240 | A | N | C | N | A | 190 | P | N | ? | 26 | - | - | - | | | | | 1500 | 250 | 1200 | |
| 113 | A | 8 :10/04/85 | 14 | 250 | A | N | C | N | A | 70 | | Y | 10 | 25 | WL1 :2 HR | 6 | 5 | | 120 | 50 | | 2200 | 220 | 1400 | HEAVY PPT IN LAST 2 MONTHS NOTED. |
| UNEXPECTED RAPID FAILURES | | | | | | | | | | | | | | | | | | | | | | | | | |
| 42 | E | 9 :05/05/71 | 27 | 260 | A | Y | T | Y | A | | | S | | MELT | SRVY: | - | | | | | | LARGE: | | 2000 | NO WARNING. CLAYEY TILL & PORE WATER PRESSURES. |

| | | GEOMETRY | | | | FOUNDAT'N | | | | | WEATHER | | | | PEAK | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| EVEN: | | TOE: | DUMP | PROF: | ? | CONDIT'NS | | ----DUMPING-- | | | | | | | DISPL | | dT | dT | dT | | | | | |
| NO | DUMP | SL- | HEI- | -ILE: | 3-D: | MAT'L: | ? | | ? | RATE | RQ: | W?: | TEMP: | PPT | MON. | FREQ. | RATE | dT TO: | PAST | PAST | PAST | B.G. | VOL. | WID-: | RUN | REMARKS |
| | CODE | DATE | OPE: | GHT | TYPE: | EFF: | TYPE | GRND: | | ACT: | BCM/M: | | | DES | (MM): | TYPE: | (M/ | FAIL. | 0.5M/: | 1 M/D: | 2 M/D: | M/DY: | LCM | TH | DUT | |
| | | | | (M) | | ECT: | | WTR | | IVE: | /DAY | | | C | | | DAY): | (HRS): | DAY | | | | *1000: | (M) | (M) | |
| FAILURES OF INACTIVE DUMPS | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 12 | C | 7 | 08/14/86 | 20 | 180 | ABA | N | | | I | | | N | | 10 | WL2 | 1 HR | 22.4 | | | >36 | | ? | 250 | 210 | WATER SEEPAGE ON FACE. INACTIVE FOR 3 MONTHS.?SNOW LAYER. |
| 14 | C | 8 | 10/05/86 | 27 | 300 | A | Y | | CT | I | | | S?: | >0 | SNOW: | - | - | | | 48 | 20 | | 115 | 750 | FAILURE DID NOT INVOLVE TOE. INACTIVE FOR 4 MONTHS. |
| 149 | C | 13 | 06/16/86 | 20 | 250 | A | N | F-CT | Y | I | | G | N | 20 | NIL | - | - | | | | | | LARGE: | FULL: | | |
| SLIVER FAILURES | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 124 | B | 1 | 08/11/86 | 15 | 220 | A | N | CT | | A | | G | N | 20 | MIN | WL2 | 2 HR | 7-24 | | | | | | 600 | | SERIES OF SLIVERS(TRIGGERED BY HEAVY DUMPING?) |
| 117 | B | 5 | 05/01/84 | 24 | 220 | A | N | CT | | A | | P | Y | >0 | MELT: | WL2 | 2 HR | 24 | 80 | | 62 | 29 | | | | SLIVER FAILURE THROUGH LAYER OF POOR QUALITY WASTE |
| 7 | C | 14 | 06/29/85 | 26 | 280 | A | N | CT | N | A | | | N | 15 | 0 | WL2 | 1 DY | | | | | | | 50 | 200 | HI-RATES & SLIVER FAILURES ALL MONTH |
| 72 | D | 1 | <APR'85 | 25 | 150 | A | Y | | | A | | | N | | | MELT: | WL2 | - | - | | | | | | | |
| 37 | E | 1 | 09/10/85 | 30 | 165 | ABC | N | C | | A | | | Y | 12 | 30 | SRVY: | 1 DY | 1.2 | | 24 | <24 | | 0.1 | 10 | 60 | |
| 32 | D | 1 | 01/13/86 | 31 | 125 | A | N | C | N | A | | | N | <0 | | - | - | | | | | | | | | |
| 88 | C | 6 | 06/22/72 | 29 | 130 | A | N | | | A | | | N | 20 | | WL2 | 1 DY | | | | | | | | | SPATIAL VARIATONS OF DISPLACEMENT RATES AVAILABLE. |
| CREEP FAILURE | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 2 | C | 6 | 05/08/83 | 20 | 200 | ? | N? | | | A | | P | S | 11 | 19 | WL2 | 2 HR | 10.6 | | 90 | 65 | | 0 | 140 | 36 | FAILURE DID NOT INVOLVE FNDN |
| 29 | C | 6 | 07/02/74 | 29 | 110 | ? | N | CT | | A | | | N | 17 | 40 | WL2 | 1 DY | 2.1 | | 96 | 72 | | 0.3 | | | |
| 27 | C | 6 | 11/12/74 | 29 | 110 | ABA | N | Q | | A | | | N | 0 | 50 | WL2 | 1 DY | 3.8 | | 192 | 120 | | 0.4 | 750 | 250 | ORGANIC FOUNDATION SOILS |
| 9 | C | 6 | 01/30/84 | 11 | 260 | A | N | F-S | | A | 60 | | N | -6 | 29 | WL2 | 2 HR | 27 | | | 76 | 50 | 1 | 775 | 140 | |
| 43 | E | 10 | 05/13/71 | 31 | 200 | A? | N | | | A | | P | S | | | MELT: | - | - | - | | | | | 120 | 2500 | MUDFLOW WENT AFTERWARDS. |
| 35 | E | 3 | 11/18/80 | 32 | 230 | BC | N | | | I | | | M | 10 | 30 | SRVY: | 10 DY: | 0.45 | | 0 | 0 | | <.2 | | 150 | BECAME ABNORMAL ON SEPT 13, 2 MONTHS BEFORE. |

| EVE-NT NO | DUMP CODE | MON. TYPE | FREQ. | TOE DUMP EVENT DATE | SL-OPE | HEI-GHT | FNDN-PRO-FILE | FNDN: EFF: ECT | J-D: MAT'L TYPE | CONDN: GRND WTR | ? ACT-IVE | ? BCM/N DAY | RATE dT/DAY | TO: EVENT DAYS | AQ: DEG C | W?: (MM) | TEMP: FAIL-URE | PPT dT/DAY | TO: M/DY | BCKGRD RATE M/DAY | PEAK RATE M/DAY | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| HIGH DISPLACEMENT RATES (NON-FAILURES) | | | | | | | | | | | | | | | | | | | | | | |
| 22 | C | 6 | | | | | | | | | | | | | | | | | | 0.3 | 5.3 | |
| 41 | E | 8 | SRVY | 01/04/69 | 33 | 150 | A-B | | | | | | | | | | | | | 0.01 | 0.3 | DUMP INACTIVE FOR 2 MONTHS///RAIN ON SNOW PACK. NEAR FAILURE. |
| 50 | A | 3 | WL1 | 12 HR:08/14/86 | 24 | 225 | B? | | R | Y | | | | | | | | | | 0.3 | 3.1 | A NEAR FAILURE. |
| 52 | A | 1 | WL1 | 12 HR:04/27/86 | 14 | 190 | A | N | C | N | A | 100 | | | M | 10 | 9 | 10 | | 0.6 | 1.68 | |
| 51 | A | 1 | WL1 | 12 HR:04/30/86 | 14 | 190 | A | N | C | N | A | 100 | | | Y | 10 | 30 | 1 | | 0.6 | 1.8 | |
| 53 | A | 3 | WL1 | 12 HR:05/86 | 24 | 225 | B? | | R | Y | A | 120 | | | Y | 20 | | | | 0.9 | 0.9 | HI-RATES IN LATE MAY TO EARLY JUNE. NO PEAKS. |
| 55 | A | 4 | WL1 | 12 HR:06/30/86 | 22 | 190 | | | C | | A | 75 | | | S?:Y | 20 | 40 | 15 | | 0.5 | 2.95 | EFFECT OF ADJACENT DUMP FAILURE. |
| 56 | A | 5 | WL1 | 12 HR:06/23/86 | 24 | 220 | | | C | | A | 160 | 3 | | Y | 20 | 40 | 8 | | 0.4 | 3.3 | |
| 57 | A | 6 | WL1 | 12 HR:06/29/86 | 22 | 190 | | | C | | A | 250 | | | M | 20 | 40 | 15 | | 0.1 | 1.3 | |
| 58 | A | 7 | WL1 | 12 HR:06/15/86 | 18 | 210 | | | C | | A | 210 | | | P?:Y | 20 | 40 | 0 | | 0.4 | 1.9 | |
| 60 | B | 1 | WL2 | 2 HR :04/82 | 15 | 180 | | | CT | N | A | | | | S | | 0 | 0 | | | 2.1 | RATES INCREASE IN APR-MAY. RIVER LEVELS INCR. 10%. |
| 61 | B | 1 | WL2 | 2 HR :08/82 | 18 | 170 | A | | CT | N | A | | | | N | | 17 | 6 | | | 2.1 | |
| 63 | B | 2 | WL2 | 2 HR :12/82 | 29 | 190 | A | | CT | N | A | | | | N | | 6 | 2 | | | 2.4 | BACKGRND RATES INCR. FROM .2 TO .5 M/DAY. |
| 65 | B | 3 | WL2 | 2 HR :04/83 | 30 | 100 | A | Y | CR | N | A | | | | S? | | 5 | 4 | | | 2.1 | DUMP JUST STARTED. |
| 66 | B | 3 | WL2 | 2 HR :06/83 | 20 | 180 | A | | CT | N | A | | | | N | | 0 | 0 | | | 0.6 | |
| 68 | B | 3 | WL2 | 2 HR :08/83 | 20 | 200 | A | | CT | N | A | | | | N | | 0 | 0 | | | 0.65 | INTRESTING CUMUL. DISPLACEMENT PLOTS. |
| 114 | B | 3 | WL2 | 2 HR :10/83 | 22 | 230 | A | | CT | N | A | | | | N | | 27 | 4 | | | 0.6 | SHORT PEAKS BUT OVERALL TREND IS NOT SEVERE. |
| 115 | B | 4 | WL2 | 2 HR :02/84 | 18 | 220 | A | | CT | N | A | | | | N | | 0 | 0 | | | 0.6 | GENERALLY HIGHER RATES. |
| 116 | B | 5 | WL2 | 2 HR :04/84 | 12 | 220 | A | | CR | N | A | | | | S | | 0 | | | | 0.7 | POOR MAT'L AND CONCENTRATED DUMPING. PRECEDES FAILURE. |
| 118 | B | 5 | WL2 | 2 HR :06/84 | 27 | 180 | A | | CT | N | A | | | | S | | 0 | | | | 0.75 | |
| 119 | B | 6 | WL2 | 2 HR :07/84 | 30 | 90 | A | | CR | N | A | | | | N | | 16 | 2 | | | 1.5 | |
| 120 | B | 6 | WL2 | 2 HR :09/84 | 25 | 220 | A | | CR | N | A | | | P | N | | 0 | | | 0.8 | 3.4 | ?TOE SUPPORT/?POOR R.Q/?FAILURE IN FACT |
| 121 | B | 7 | WL2 | 2 HR :05/85 | 22 | 220 | A | | CR | N | A | | | | S | | 0 | | | | 1.5 | SOME CRACKS. |
| 122 | B | 8 | WL2 | 2 HR :06/85 | 30 | 100 | A | | CR | N | A | | | | N | | 0 | | | | 0.9 | LIFTING DUE TO HIGH SETTLEMENT. |
| 123 | B | 9 | WL2 | 2 HR :06/85 | 25 | 100 | A | | CR | N | A | 125 | | | N | | ? | ? | | | 0.75 | MIDDLE STAGE OF WRAP-AROUND. |
| 90 | C | 1 | WL2 | 2 HR :06/12/86 | 23 | 240 | A | N | CT | N | A | 25 | | | N | 16 | 11 | 3 | | 0.05 | 1.9 | RATES INC. QUICKLY AFTER MONITORS RESET. |
| 91 | C | 2 | WL2 | 2 HR :03/29/86 | 23 | 220 | A | N | CT | N | A | 90 | | | M | 0 | 19 | 0 | | 0.25 | 3.4 | HI-DUMPING RATES IN ADJACENT REGION AFFECTING RATES. |
| 92 | C | 2 | WL2 | 2 HR :05/30/86 | 23 | 220 | A | N | CT | N | A | 15-60 | | | Y | 20 | 30 | 7 | | 0.35 | 2.25 | MONITORS REACT WITHIN A FEW DAYS TO TEMP. RISE. |
| 93 | C | 2 | WL2 | 2 HR :06/12/86 | 23 | 220 | A | N | CT | N | A | 80 | | | N | 16 | 11 | 3 | | 0.05 | 2.55 | RATES INCREASE QUICKLY AFTER MONITORS RESET. |
| 94 | C | 2 | WL2 | 2 HR :08/25/86 | 23 | 220 | A | N | CT | N | A | 40-70 | | | N | 20 | 0 | | | 0.1 | 4.2 | |
| 95 | C | 3 | WL2 | 2 HR :03/05/86 | 24 | 245 | A | N | CT | N | A | | | | N | -4 | SNOW | | | 0.3 | 0.7 | |
| 96 | C | 3 | WL2 | 2 HR :03/26/86 | 24 | 245 | A | N | CT | N | A | 15 | | | N | 0 | 0 | | | 0.2 | 1.35 | HI-DUMPING RATES IN ADJACENT REGION AFFECTING RATES. |
| 97 | C | 3 | WL2 | 2 HR :04/18/86 | 24 | 245 | A | N | CT | N | A | 100 | | | N | 0 | MIN. | | | 0.4 | 0.75 | |
| 98 | C | 3 | WL2 | 2 HR :05/29/86 | 24 | 245 | A | N | CT | N | A | 15-60 | | | Y | 20 | 30 | 8 | | 0.3 | 2.15 | MONITORS REACT W/N A FEW DAYS TO TEMP. RISE. |
| 99 | C | 3 | WL2 | 2 HR :08/25/86 | 23 | 260 | A | N | CT | N | A | 40 | | | N | 20 | 0 | | | 0.2 | 2.45 | |
| 100 | C | 3 | WL2 | 2 HR :09/22/86 | 23 | 260 | A | N | CT | N | A | 80 | | | M | 15 | 38 | 20 | | 0.55 | 1.6 | |
| 101 | C | 5 | WL2 | 2 HR :03/15/86 | 32 | 200 | A | N | CT | N | A | 50 | | | Y | 4-10 | MIN. | | | 0.4 | 1.4 | DISPL. RATES PEAK 3 DAYS AFTER TEMP. HITS 10 DEG |
| 102 | C | 5 | WL2 | 2 HR :05/27/86 | 32 | 210 | A | N | CT | N | A | 15 | | | Y | 10 | 30 | 2 | | 0.2 | 1 | ADJACENT DUMP IS ACTIVE. |
| 103 | C | 5 | WL2 | 2 HR :06/09/86 | 32 | 210 | A | N | CT | N | A | 30 | | | M | 20 | 20 | 0 | | 0.1 | 3.7 | ADJACENT DUMP IS ACTIVE. |
| 104 | C | 5 | WL2 | 2 HR :09/23/86 | 29 | 225 | A | N | CT | N | A | 60 | | | N | | MIN. | | | 0.05 | 1.6 | |
| 105 | C | 5 | WL2 | 2 HR :10/22/86 | 28 | 230 | A | N | CT | N | A | 120 | | | N | | MIN. | | | 0.2 | 1.25 | |
| 106 | C | 5 | WL2 | 2 HR :11/24/86 | 26 | 235 | A | N | CT | N | A | 120 | | | N | | MIN. | | | 0.2 | 1 | |
| 107 | C | 15 | WL2 | 2 HR :09/05/86 | 30 | 250 | A | N | CT | N | A | 110 | | | Y | 20 | 39 | 7 | | 0.1 | 1.4 | |
| 108 | C | 15 | WL2 | 2 HR :09/27/86 | 30 | 265 | A | N | CT | N | A | 60 | | | N | | MIN. | | | 0.1 | 1.9 | |
| 109 | C | 15 | WL2 | 2 HR :11/24/86 | 28 | 290 | A | N | CT | N | A | 120 | | | N | | MIN. | | | 0.2 | 1 | A SERIES OF DISPL. PEAKS DURING PERIOD. |
| 71 | D | 1 | WL2 | 2 HR :06/30/86 | 32 | 220 | A | N | CT | N | A | | | | Y | 20 | SOME | | | 0.3 | 2.5 | NO DATA BEFORE MAY30 |
| 110 | D | 1 | WL2 | 2 HR :01/28/86 | 31 | 125 | A | N | CT | N | A | | | | N | -10 | SNOW | | | 0.2 | 2.1 | |
| 111 | D | 1 | WL2 | 2 HR :12/85 | 31 | 125 | A | N | CT | N | A | | | | N | <0 | COLD | | | | | |

| EVE-NT NO | DUMP CODE | MON. TYPE | FREQ. | TOE DUMP EVENT DATE | HEI-GHT (M) | FNDN: PRO-FILE | 3-D EFF-ECT | CONDN: MAT'L TYPE | ? GRND WTR | ? ACT-IVE | RATE dT/DAY | TO:RQ DAYS | W?: EVENT | TEMP: DEG C | PPT (MM) | dT TO: FAIL-URE | BCKGRD RATE M/DY | PEAK RATE (M/DAY | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 125 | G 1 | TVS | 1-7DY:07/77 | :20 | 80 | A | N | CT | N | A | | | N | | | | | 0.65 | VISIBLE TOE SPREADING. IMMEDIATELY AFTER COMPLETION. |
| 126 | G 2 | TVS | 1-7DY:02/83 | :30 | 100 | A | N | C | Y | A | | | P N | | | | 0.02 | 0.1 | SNOW WITHIN FILL DURING CONSTRUCTION. |
| 128 | G 2 | TVS | 1-7DY:06/83 | :25 | 150 | A | N | C | Y | A | | | N | | | | 0.04 | 0.075 | PEAK AT SPRING THAW. TOE BERM BEING CONSTRUCTED. |
| 129 | G 2 | TVS | 1-7DY:07/83 | :25 | 150 | A | N | C | Y | A | | | Y | | 40 | | 0.03 | 0.1 | PEAKS RESPOND TO 25-50 mm RAIN. TOE BERM NEAR COMPLETE. |
| 132 | G 2 | TVS | 1-7DY:05/84 | :25 | 150 | A | N | C | Y | A | | | S | | | | 0.01 | 0.03 | ROAD WIDENING. RESPONSE TO SPRING THAW. |
| 134 | G 2 | TVS | 1-7DY:11/85 | :25 | 150 | A | N | C | Y | A | | | Y | | | | 0.002 | 0.01 | MINIMAL RESPONSE TO PPT. |
| 135 | G 3 | TVS | 1-7DY:03/85 | :25 | 75 | A | N | C | Y | A | | | Y | 30 | 6 | | 1 | 25 | SPRING THAW,SUMMER AND FALL STORMS. |
| 136 | G 3 | TVS | 1-7DY:06/85 | :25 | 75 | A | N | C | Y | A | | | N | | | | 0.005 | 0.05 | TOE SPREADING AND FACE BULGE. |
| 137 | G 3 | TVS | 1-7DY:08/85 | :25 | 75 | A | N | C | Y | A | | | N | | | | 0.003 | 0.05 | RESPONSE TO 30 mm RAIN. |
| 138 | G 3 | TVS | 1-7DY:10/85 | :25 | 75 | A | N | C | Y | A | | | N | | | | 0.003 | 0.008 | TOE SPREADING AND FACE BULGE. 5m OF CREST SETTLEMENT. |
| 139 | G 3 | TVS | 1-7DY:12/85 | :25 | 75 | A | N | C | Y | A | | | N | | | | | | |
| 140 | G 3 | TVS | 1-7DY:02/86 | :20 | 100 | A | N | C | Y | A | | | N | 35 | 6 | | | 0.1 | |
| 142 | F 1 | TVS | 30 DY:05/84 | :26 | 70 | A | N | CT | N | I | 90 | | S | | | | 0.005 | 0.05 | SPRING THAW. INACTIVE SINCE JAN'84 |
| 143 | F 1 | TVS | 30 DY:05/85 | :26 | 70 | A | N | CT | N | I | 450 | | S | | | | 0.001 | 0.004 | SPRING THAW. |
| 145 | F 2 | TVS | 30 DY:12/86 | :17 | 80 | A | N | CT | N | I | | | N | | | | 0.001 | 0.003 | MINOR RESPONSE TO RAIN EVENTS. INACTIVE OCT'84. |
| 146 | F 3 | TVS | 30 DY:12/86 | 0 | 28 | A | N | CT | N | I | | | N | | | | 0.01 | 0.035 | INACTIVE APR'85. PEAK IN JUNE. STEEP VECTOR AT CREST. |
| 147 | F 4 | TVS | 30 DY:12/86 | :10 | 45 | A | N | CT | N | I | 240 | | N | | | | 0.005 | 0.01 | INACTIVE AUG'84. PEAK IN APR/MAY. |
| 148 | F 5 | TVS | 30 DY:03/87 | 0 | 20 | A | N | PF | N | A | | | S | | | | 0.002 | 0.005 | IN-PIT DUMP. PEAK IMMEDIATELY FOLLOWING CONSTRUCTION. |
| 149 | F 6 | TVS | 30 DY:04/87 | :15 | 38 | A | N | WR | N | A | | | S | | | | 0.015 | 0.065 | PEAK IMMED. AFTER CONSTRN. OVERLIES EXISTING DUMP. |
| 150 | C 16 | WL2 | 2 HR :02/21/87 | 0 | 110 | A | N | PF | N | A | | | P N | | | | 0.2 | 2.5 | POOR QUALITY WASTE CAUSING QUICK DISPLACEMENT PEAKS. |
| 151 | H 1 | TVS | 30 DY:11/86 | :20 | 180 | A | N | CT | Y | I | | | P Y | | | | 0.02 | 0.16 | DUMP REACTS TO RAINFALL EVENTS. |

NORMAL ACTIVITY AND SPECIAL MONITORING.

| | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 72 | D 1 | WL2 | :03/01/87 | :30 | 220 | A | | C | N | A | | | N | COLD | | | 0.3 | 1 | MORE STABLE THAN SAME PERIOD A YEAR AGO. |
| 85 | E 2 | SRVY | :10/11/84 | :20 | 190 | A | | C | N | I | | | Y | MELT | | | 0.005 | 0.035 | HIGHEST RATES IN MAY NEAR OLD FAILURE |
| 33 | E 1 | SRVY | :02/24/86 | :20 | 150 | A | | C | N | I | | | P N | COLD:SNOW | | | | 0.7 | DUMP PROFILE AFTER 7 MONTHS OF INACTIVITY. |
| 86 | E 3 | SRVY | :03/22/86 | :10 | 195 | A | | C | N | A | 200 | | N | | | | | 0.38 | |
| 84 | E 2 | SRVY | :03/18/85 | :20 | 150 | A | | C | N | A | | | N | | | | 0.003 | 0.01 | |
| 34 | E 2 | SRVY | :03/09/86 | :30 | 170 | A | | C | N | A | 430 | | V Y | | 60 | 3-6 | 0.1 | 2.5 | MAR 1 IS LAST DAY OF A PUSH TO TOE INTO GULLEY |
| 82 | E 2 | SRVY | :02/27/87 | :19 | 210 | A | | C | N | A | | | N | | | | | | DUMP PROFILE AFTER 2 WEEKS OF INACTIVITY |
| 83 | E 2 | SRVY | :01/07/87 | :20 | 130 | A | | C | N | A | | | N | | | | | | VECTORS & SPATIAL VARIATIONS. |
| 87 | E 4 | SRVY | :03/01/87 | :10 | 190 | A | | C | N | A | | | N | | | | | | VECTOR MEASUREMENTS. |
| 127 | G 2 | TVS | 1-7DY:03/83 | :30 | 100 | A | N | C | Y | A | | | S | | | | 0.03 | | NO CONSTRUCTION. |
| 130 | G 2 | TVS | 1-7DY:11/83 | :25 | 150 | A | N | C | Y | A | | | N | | | | 0.02 | | ADDITIONAL BERM CONSTRUCTION. |
| 131 | G 2 | TVS | 1-7DY:03/84 | :25 | 150 | A | N | C | Y | A | | | N | | | | 0.01 | | NO ACTIVITY. |
| 133 | G 2 | TVS | 1-7DY:10/84 | :25 | 150 | A | N | C | Y | A | | | N | | | | 0.015 | | ROAD WIDENING. MINIMAL RESPONSE TO PRECIP. |
| 141 | G 3 | TVS | 1-7DY:04/86 | :20 | 100 | A | N | C | Y | A | | | S | | | | 0.04 | | CRACKS OPEN HORIZONTALLY. FACE BULGE. |
| 144 | F 1 | TVS | 30 DY:12/86 | :26 | 70 | A | N | CT | N | I | | | G N | | | | 0.001 | | LONGTERM CREEP. |

# APPENDIX C

# LABORATORY MODEL TEST

# APPENDIX C

## NOTES REGARDING
## THE FAILURE MECHANISM OF A MODEL DUMP

Subtle patterns that have been observed on the faces of waste rock dumps provide an indication of the strains that develop within the body of a dump, and provide an indication of the mechanism that governs dump stability. The strain patterns that have been observed on the faces of waste rock dumps have been interpretated as an indication that dump stability is controlled by a double wedge mechanism. The toe wedge provides support for the active wedge which, because of the strains that take place within the toe region of the dump, is in the active Rankin state. Consequently, the shear strength of the mass of material within the active wedge is fully mobilized. This mechanism is discussed in a paper titled 'Stability and Performance of Waste Dumps on Steeply Sloping Terrain' by D.B. Campbell. This paper was presented at, and was published in the Proceedings of, the International Symposium on Geotechnical Stability in Surface Mining, held at Calgary, Alberta, 6-7 November, 1986.

The double wedge mechanism has been interpreted on the basis of strain patterns observed on dump faces. However, there is never an opportunity to examine the interior of a full scale dump to provide data or information whereby the double wedge mechanism can be confirmed.

In order to examine the deformations that take place in the interior of a waste rock dump, a model waste dump was constructed in the laboratory. Lateral confinement for this model dump was provided by a glass plate that forms a vertical plane aligned parallel to the fall line on the face of the model dump. The glass plate permits the model

dump to be viewed in cross section. The toe region was supported by a structural plate aligned perpendicular to the base of the model dump, and equipped with a screw mechanism to permit controlled displacement of the dump toe.

The material comprising the model dump consisted of clean, cohesionless, fine to medium sand which in the loose state exhibits an angle of internal friction of 31°, as indicated by the angle of repose of the sand on the face of the model dump.

A layer of micro-balloons was introduced at the base of the model dump. The micro-balloons consist of minute plastic spheres, which exhibit an angle of internal friction of approximately 22 degrees. The layer of micro-balloons resulted in a base friction angle that is significantly lower than the angle of internal friction for the sand comprising the model dump, and simulates the conditions for a full scale prototype dump where the friction angle of the soil mantle at the dump/foundation interface is lower than the angle of internal friction for the waste rock comprising the full scale waste rock dump.

A series of horizontal lines consisting Ottawa sand where introduced at the interface between the lateral limit of the model and the inside surface of the confining glass plate. These white horizontal lines within the dump cross section served as references, so that as the toe region of the dump was permitted to yield, the resulting deformations within the body of the dump could be traced. The deformations that occurred within the body of the dump as the toe was permitted to yield down slope were recorded by a series of photographs marked 1 to 6 inclusive. Photograph No. 1 represents conditions before any displacement of the toe had taken place, and Photograph No. 6 represents conditions after the dump toe had moved through a distance of approximately 40 millimetres in a direction parallel to the base. For reference and

scale, the surface of the glass plate was marked in 100 millimetres squares. This reference grid is clearly visible on the photographs.

Photograph No. 6 shows the section through the model dump after the toe region had moved outward a distance of approximately 40 millimetres. This displacement of the toe region of the dump resulted in development of a wedge - shaped failure zone below the dump face. The back of the failure wedge, which consists of a narrow plane extending from the upper limit of the inclined dump face to the base of the model dump is inclined at 56° to the horizontal. The front boundary of the failure wedge, which is also sensibly planer, but which is represented by a somewhat broader band, is inclined at approximately 5° from the vertical. The inclinations of the wedge boundaries that are clearly evident in the photo record for the model dump are in close agreement with the critical angles as indicated by our double wedge stability analyses.

The observed deformations within the model dump indicate that the assumption that the boundaries at both the back and the front of active wedge are planar is valid. The deformations within the region of the bottom apex of the active wedge in the model dump do not show any indication of radial shear deformations forming a transition zone between the shear displacements at the back of the active failure wedge (inclined at 56°), and the sheer displacements along the base, (inclined at 30°).

Within the boundaries of the active wedge, the vertical spacing between the bands of Ottawa sand had decreased by approximately 3 per cent by the time that the toe displacement had reached 6 mm, and the spacing between the horizontal bands remained constant as the displacements increased from 6 to 40 mm. This reduction in the vertical dimension within the active wedge was undoubtedly the result of the vertical contraction that accompanied horizontal expansion of the active wedge in response to relief of lateral stress with the initial movement of the

toe. Small movements of the toe could be expected to result in active Rankine stress conditions within the active wedge, and these stress conditions would have remained essentially constant as the toe movements increased. Consequently, the vertical contraction within the active wedge could be expected to occur at or near the outset of the toe displacements, and no further contraction would be expected to develop as the toe displacements increased. The series of photographs taken at the increments of toe displacement as indicated, shows that this in fact was the case.

At the outset of the test, i.e. for zero displacement the toe region of the model dump, the lines formed by the bands of Ottawa sand were horizontal. After the toe of the model dump had displaced 40 millimetres, the inclination of these bands of Ottawa sand within the active wedge were inclined at approximately 5°, sloping upward toward the dump face. Considering that the boundaries of the active wedge are planar, this change in the inclination of the horizontal bands of Ottawa sand extending across the active wedge could not have resulted from rotation about a central point. The change in the inclination of the horizontal bands must have been the result of linearly distributed differential shear across the active wedge. This demonstrates that the sand within the active wedge was truly in the active Rankin state.

In summary, the photographic records of the model dump experiment show the following:

- The double wedge mechanism is a valid model.
- The wedge boundaries are planar.
- Modest strains within the region of the dump toe result in active Rankine stress conditions within the active wedge. This implies that the shear strength parameters of the materials within the active wedge, as well as at the wedge boundaries are fully mobilized.

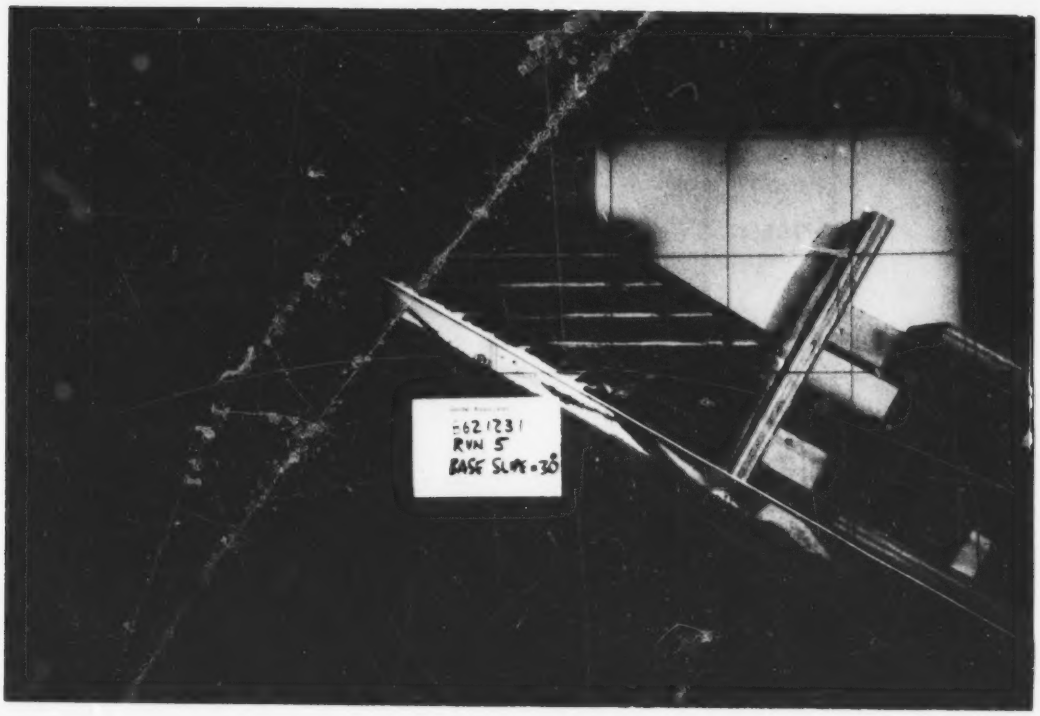



PHOTO #1, TOE DISPLACEMENT = 0mm

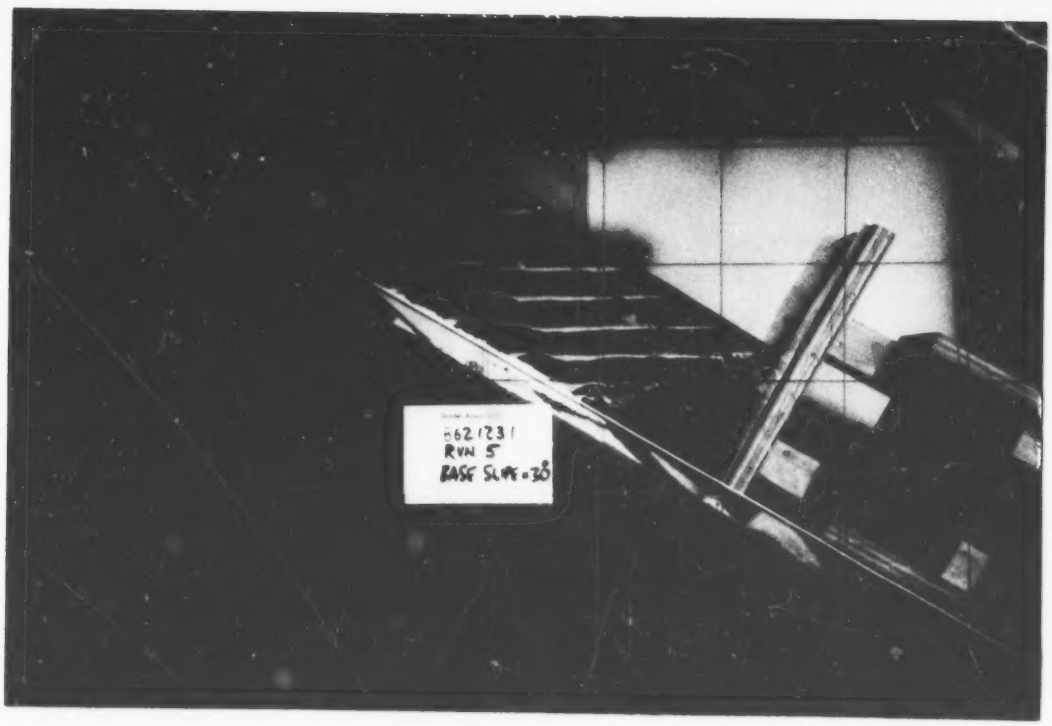



PHOTO #2, TOE DISPLACEMENT = 7mm

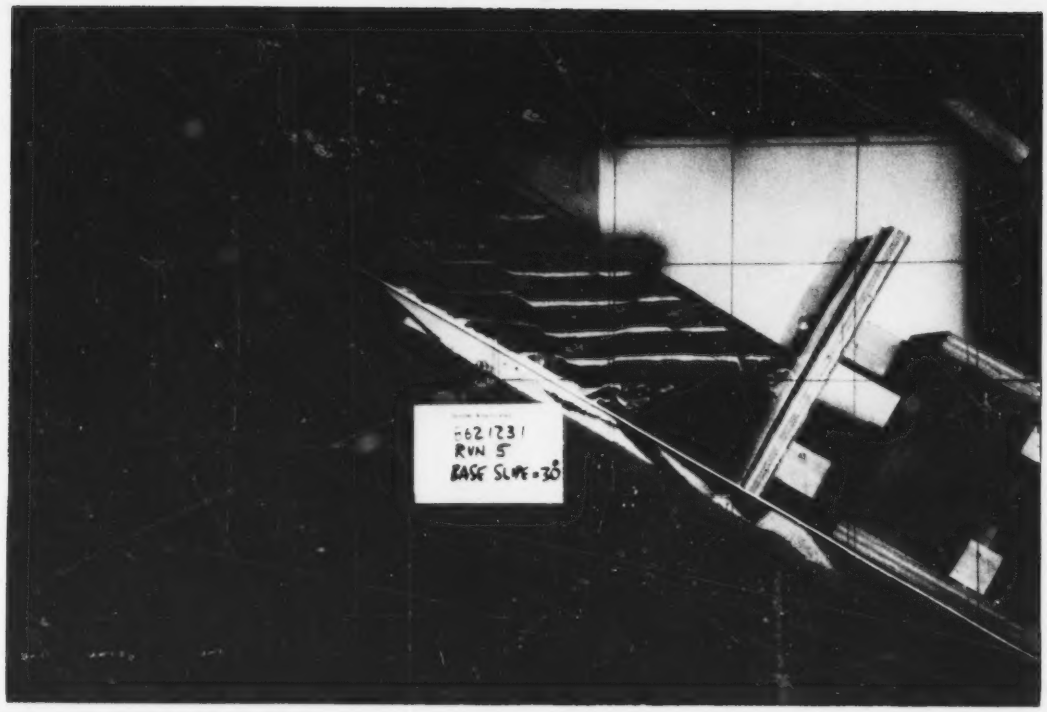



PHOTO #3, TOE DISPLACEMENT = 12mm

PHOTO #4, TOE DISPLACEMENT = 19mm

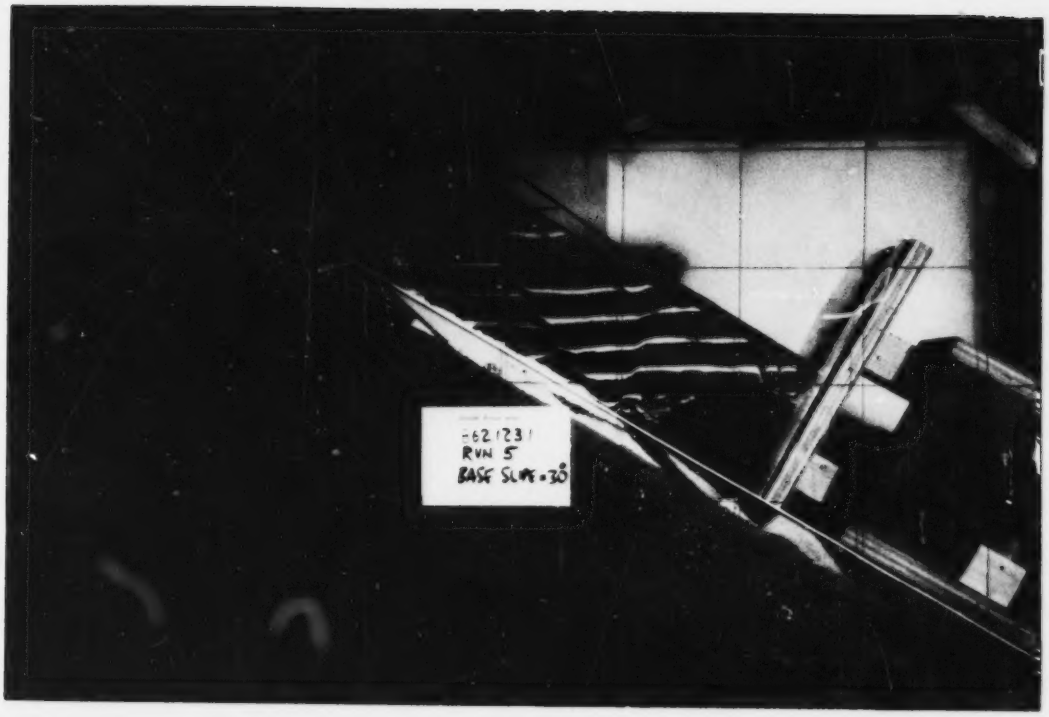
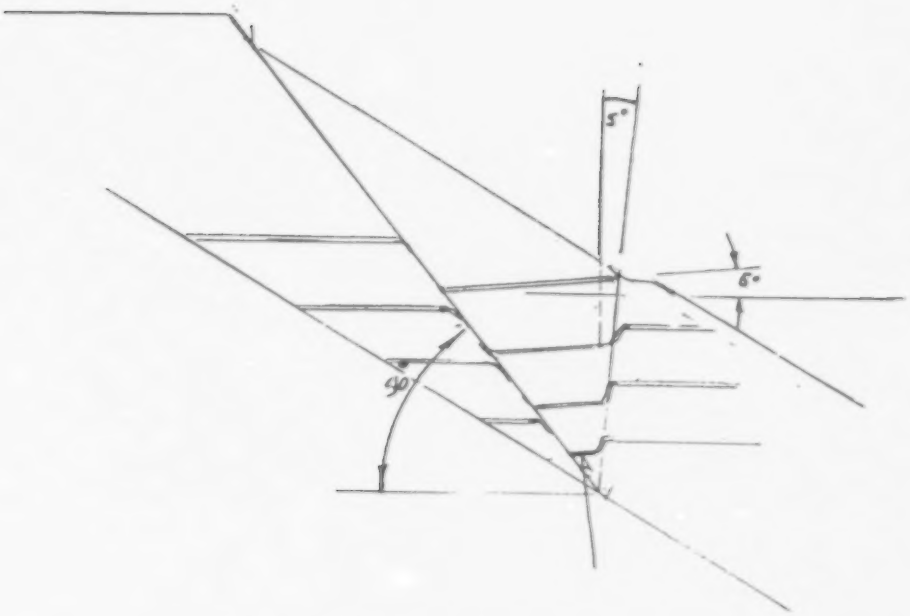



PHOTO #5, TOE DISPLACMENT = 26mm

PHOTO #6, TOE DISPLACMENT = 40mm

# APPENDIX D

# RUNOUT ANALYSIS




WITHIN THE BOUNDARIES OF THE ACTIVE WEDGE, THE VERTICAL SPACING BETWEEN THE BANDS OF OTTAWA SAND HAD DECREASED BY APPROXIMATELY 3% BY THE TIME THAT THE TOE DISPLACEMENT HAD REACHED 6mm, AND THE SPACING BETWEEN THE HORIZONTAL BANDS REMAINED CONSTANT AS THE DISPLACEMENTS INCREASED FROM 6 TO 40mm. THIS REDUCTION IN THE VERTICAL DIMENSION WITHIN THE ACTIVE WEDGE WAS UNDOUBTEDLY THE RESULT OF VERTICAL CONTRACTION THAT ACCOMPANIED HORIZONTAL EXPANSION OF THE ACTIVE WEDGE IN RESPONSE TO RELIEF OF LATERAL STRESS WITH THE INITIAL MOVEMENT OF THE TOE. SMALL MOVEMENTS OF THE TOE COULD BE EXPECTED TO RESULT IN ACTIVE RANKINE STRESS CONDITIONS WITHIN THE ACTIVE WEDGE, AND THESE STRESS CONDITIONS WOULD HAVE REMAINED ESSENTIALLY CONSTANT AS THE TOE MOVEMENTS INCREASED. CONSEQUENTLY, THE VERTICAL CONTRACTION WITHIN THE ACTIVE WEDGE COULD BE EXPECTED TO OCCUR AT OR NEAR THE OUTSET OF THE TOE DISPLACEMENTS, AND NO FURTHER CONTRACTION WOULD BE EXPECTED TO DEVELOP AS THE TOE DISPLACEMENTS INCREASED. THE SERIES OF PHOTOGRAPHS TAKEN AT APPROXIMATELY 6mm INCREMENTS OF TOE DISPLACEMENT SHOW THAT THIS IN FACT WAS THE CASE.

AFTER THE TOE HAD DISPLACED APROXIMATELY 40mm, THE SEGMENT OF THE BANDS OF OTTAWA SAND EXTENDING THROUGH THE ACTIVE WEDGE WERE INCLINED AT 5 DEG, SLOPING UPWARD TOWARD THE FACE OF THE MODEL DUMP. SINCE THE BOUNDARIES OF THE ACTIVE WEDGE ARE PLANAR, THIS CHANGE IN THE INCLINATION OF THE HORIZONTAL BANDS COULD NOT HAVE BEEN THE RESULT OF ROTATION: IT MUST BE THE RESULT OF DIFFERENTIAL SHEAR DISPLACEMENTS THAT WERE LINEARLY DISTRIBUTED WITH RESPECT TO HORIZONTAL DISTANCE ACROSS THE ACTIVE WEDGE.

IN SUMMARY, THE PHOTOGRAPHIC RECORDS OF THE MODLE DUMP EXPERIMENT SHOW THE FOLLOWING:

- THE DOUBLE WEDGE MECHANISM IS A VALID MODLE.
- THE WEDGE BOUNDARIES ARE PLANAR
- MODEST STRAINS WITHIN THE REGION OF THE DUMP TOE RESULT IN ACTIVE RANKINE STRESS CONDITIONS WITHIN THE ACTIVE WEDGE. THIS IMPLIES THAT THE SHEAR STRENGTH PARAMETERS OF THE MATERIALS WITHIN THE ACTIVE WEDGE, INCLUDING AT THE WEDGE BOUNDARIES ARE FULLY MOBILIZED.

# APPENDIX D

## ANALYSIS OF WASTE DUMP FAILURE RUNOUT

### 1.0 CHARACTERISTICS OF WASTE DUMP FAILURES

The following notes describe types of failure which have been observed in waste dumps under conditions similar to those expected at the proposed dump sites.

Crest Sliver Failure : oversteepening of dump crest due to high rate of dumping and presence of fines may cause a sliver to slump and slide down the face of the dump. Rate of movement down face is relatively slow and impact at base of dump is minor.

Dump Face Slump : Spreading of foundation soils beneath the toe is usually relatively localised and causes a sheet or a layer of waste rock on the dump face to slump. As for sliver failures, most of slump mass is supported on underlying waste rock with high frictional resistance to sliding. Thus slump does not accelerate towards toe.

Deep-seated Failure : Mass movement of dump can occur if quasi-continuous plane of weakness present at or below dump-foundation contact. This may be due to unfavorable orientation of bedrock discontinuities, lightly consolidated soils and/or development of excess pore water pressures. As displacement of slide mass occurs de-stabilising moments and forces tend to decrease and rate of movement also declines.

Steep Foundation : High stresses occur in the toe region of dumps resting on steep slopes in excess of about 25 degrees. In response, large

deformations occur in the face of the dump and also a relatively high degree of strain energy is stored as a result of the elastic deformation occurring at contacts between waste rock particles. Face may oversteepen to point where instability occurs - if slope below is steep the slide mass may accelerate.

Sturzstrom Event : As for the Steep foundation case above, failure of a number of point contacts in highly stressed zones may cause a rapid release of stored strain energy in the toe region of the dump. One observer of such an event described movement of particles in the toe region as "explosively outwards".

## 2.0 OBSERVED NATURE OF WASTE DUMP FAILURE RUNOUT

We have studied available data from our experiences with past waste dump projects under similar conditions. A total of 12 events have been examined. Cross-sections of the run-out paths and details of analyses of the kinematics of each event are contained in this Appendix.

To preserve confidentiality we have omitted the location and year of occurrence of all events. All of the events occurred at operating coal mines in B.C. located in mountainous terrain.

The principal characteristics of each event are summarised in Table D-1. The dump heights ranged from 150m to 240m, with the toe region supported on slopes exceeding 25 degrees in all cases of dynamic debris movement.

Volume of slide per unit width of failed crest has been calculated as a means of comparing the magnitude of each event. Where field esti-

mates of slide geometry and volume are not available we have assumed an Active-Passive wedge mechanism as shown on Figure D-1.
Normalised slide volumes range from 2000 to 7000 cubic metres per metre of crest.

We consider that it is appropriate to divide the run-out area into at least two segments. The initial segment, located immediately below the toe of the dump, was generally inclined at angles in the range 15 to 25 degrees. This segment is referred to as the acceleration or 'run-in' zone over which it appears that during many of the slide events the front of the slide either accelerated or retained significant momentum. Beyond the accelleration zone we have defined the 'run-out' zone which is inclined at less than 15 degrees. In fact, the run-out zone is usually flatter than 10 degrees over much of its length. We have obtained only three reports of observations of actual failure events. One of these was event C3 which moved at a brisk walking pace of 1 to 2 m/s. Another event not documented but similar to event A1 resulted in the aforementioned explosive behaviour of the toe. From the height of a spur cleared by that slide we have deduced that the debris was travelling at a velocity in excess of 40 m/s. Elsewhere estimates of the degree of dynamic super-elevation during travel around obstructing terrain or of run-up of opposing valley slopes have provided an estimate of velocity at some stage of the run-out, as shown on Table D-1. These back analyses indicate that velocities in the range 10-25 m/s have occurred. An eyewitness account of an event similar to event C8 on Table D-1 describes velocities in the range 9-13 m/s (20-30 miles per hour).

For events B1, D2 and D5 slide debris travelled over a steep acceleration zone and impacted on the opposite valley slope at the beginning of the run-out zone. Slide debris in event B1 is estimated to

have been deflected through an angle of 60 to 70 degrees at the point of entry to the run-out zone.

In event D2 slide debris ran up the opposite valley slope to an elevation approximately 20 m above the creek. Only a small portion of the debris reaching the run-up area was deflected a relatively small distance down the valley.

In contrast, debris from event D5 travelled over a relatively steep side slope and impacted on the floor and opposite slope of the valley. The direction of the acceleration zone was approximately perpendicular to the axis of the valley. As a result of this impact, waste rock and overburden soil spread laterally in a downstream direction over a distance of about 1,000 m on a 7 degree slope. We consider that shear resistance to sliding in the downstream direction was reduced by excess pore water pressure generated as a result of the slide debris impacting upon the loose saturated soil on the valley floor. The slide debris would decelerate as pore water pressures at the base diminish, the thickness of the debris flow decreased, and the base gradient decreased.

## 3.0 OBSERVED NATURE OF SNOW AND ROCK AVALANCHES

We have reviewed reports of various types of snow and soil or rock avalanches with the purpose of identifying similarities in behaviour and developing applicable methods of predicting run-out.

### Snow Avalanches

Considerable research has been devoted to analysis of snow avalanches in Canada, the U.S. and Europe in the last ten years. Perla, Cheng and McLung (1980) proposed a rational model which sought to separately incorporate factors contributing to deceleration of snow avalanches. Previous methodology was of a highly empirical nature based upon field

observations with a limited understanding of contributing physical processes. The analytical model developed by Perla et al required assessment of two parameters tending to cause deceleration: base friction and aerodynamic drag. Bakkehoi, Domaas, and Lied (1983) applied this concept in a regression analysis of 206 avalanches in order to derive best-fit relationships between contributing factors and in the relative values of frictional and aerodynamic drag.

Although the physical properties of snow are very different from waste rock, the approach of testing and adjusting a theoretical physical model against numerous documented events is logical. Further the concept of a model which deals with the progression of material along the run- out path in a step wise manner is reasonable. The bulk density of waste rock is much greater than the density of avalanching powder snow. Thus aerodynamic drag is not expected to be significant in decelerating waste rock debris slides.

It is significant that Bakkehoi et al found that snow avalanches consistently accelerate on steep slopes and slow down on flatter slopes, confirming that base friction is a major component of sliding resistance. Dry snow avalanches are reported to achieve peak velocities of up to 50-100 m/s. Wet snow avalanches reach lower peak velocities but tend to travel further over slopes of less than 10 degrees. Dry snow slides decelerate and commence deposition rapidly on slopes of less than about 10 degrees.

Major Rockslides

Major rockslides and rock debris avalanches in mountainous terrain involving more than a million cubic metres have been documented and analyzed by several investigators. Eye-witness accounts of these events are non-existent, and reliable correlations between contributing factors

and measured run-out distance are not available. Eisbacher (1978) documented and assessed a number of naturally occurring major sturzstrom rock avalanches in northwestern Canada. He found a broad scatter between slide volume and run-out distance. This confirms the need for more detailed evaluation of the geometric and other physical factors involved. From analysis of run-up against obstructing topography, Eisbacher deduced velocities of between 30 and 100 m/s for major slides involving up to 30 million cubic metres of rock on slopes as steep as 30 degrees. He inferred that run-out distance may be affected by the extent and degree of collapse of the sliding mass. Subsequent slumps may add momentum impulses to the front of the slide. Eisbacher also concluded from examination of slide debris that material on the surface of the moving mass is tumbled relatively gently and that the larger rock fragments tend to be raised to the surface of the debris.

Cruden and Hunger (1986) reviewed the 1903 Frank Slide near Crowsnest Pass, Alberta. They found that the grain size of debris increases with elevation in a given cross-section. It is concluded that "dispersive pressure" - the sum of normal inter-grain forces in a rapidly shearing granular mass, drives the larger grains to the surface. Also, they found no evidence of fluidisation due to pore fluid (air) pressure.

Cruden (1985) concluded from analyses of sturzstrom events that low values of friction necessary to allow observed travel distances also result in unrealistically high velocities. Cruden suggests introduction of a component of base sliding resistance related to turbulent shear in the boundary layer between the moving debris and the foundation. Development of a reliable predictive relationship depends upon numerous well-documented events for calibration, and is not now available. We consider that this type of approach will be required to correctly analyze sturzstrom type events such as A-1 on Table D-1.

Debris Torrents

In the last five years, attention has focussed on slides involving torrents of saturated soil, rock and forest debris in steep mountain terrain. These slide events usually involve a steeply inclined run-in or acceleration zone with slopes in the range 10 to 30 degrees, and a run-out or deposition zone with slopes of less than 10 degrees.

Hungr et al (1984) developed quantitative analyses for design of protection against debris torrents.

Recorded observations of saturated debris torrents on the West Coast of B.C., Japan and Europe indicate the following general characteristics:

- the leading portion of the debris flow is usually armoured with large boulders which oscillate or rock gently without intense mixing.
- the leading portion is followed by finer grained soil and water, and several surges of coarse debris may occur.
- the flow of the leading portion can be characterized as laminar.
- debris torrents will commence deposition on slopes in the range of about 8 to 14 degrees. The lower part of this range is applicable to torrents in more confined channels while flows will deposit on relatively steep open surfaces.

Hungr et al adopted the momentum conservation equation for a steadily moving debris front developed by Takahashi and Yoshida (1979). This method is considered to be a valid approach for debris torrents which have observed or inferred velocities up to about 10 m/s upon entry to the run-out or deposition zone. The method assumes that sliding resistance is due to an equivalent value of basal friction.

## 4.0 BACK ANALYSIS OF WASTE DUMP RUN-OUT

We consider that the run-out of debris from potential failure of the proposed waste dumps can be predicted using the frictional model employed by Hungr et al for debris torrents. We have modified the equations quoted by Hungr et al to represent an unsaturated debris mass sliding on a base layer which may be saturated.

The rapid advance of waste dump slide debris onto lightly consolidated, saturated, surficial soils is likely to result in excess pore water pressures which reduce the effective shear resistance to sliding over the run-out zone.

The results of back analysis of past failure events are summarized on Table D-2. Figure D1 illustrates the geometry of the assumed runout mechanism. We have estimated velocities attained by the slide mass in the 'run-in' zone above the points where the slope flattens to less than 10 degrees. The method of estimating the run-in velocity consists of calculating kinetic energy gained due to loss of potential energy, and lost as work against sliding friction. Selection of an equivalent average angle of friction over the run-in zone required judgement based on the type of expected failure events. A sturzstrom event with a high velocity can be expected if the toe is supported on steep slopes and the indicated factor of safety for the dump is close to unity. A less dynamic slump of a dump founded on relatively flat terrain is likely to be decelerated by higher base shear stresses resulting from a higher equivalent base angle of friction.

Table D-2 summarizes the results of calculations to determine combinations of average base friction angle over the run-in or acceleration zone and the run-out zones which best fit the known boundary conditions of slope, distance travelled and inferred velocity (if available) at any point. The depth of the debris as it flows into

the run-out zone was estimated on the basis of the relative geometry of each dump assuming that the failure occurs with a back slope of about 45 degrees. This is considered to be a reasonable value based on the results of stability analyses.

The average equivalent friction angle required over the base of the run-out zone to stop the mass leaving the run-out zone within the observed distance has been calculated using the principal of momentum conservation, as outlined by Hungr et al. Where available inferred or observed velocities at certain points in the run-out zone have been incorporated into the back-analysis as additional boundary constraints.

Nearly all of the documented failures occurred on dumps supported on quasi-planar slopes inclined in excess of 25 degrees. The indicated factor of safety against base sliding on this slope is unity for an assumed friction angle of 32 degrees. In cases such as C3, D1 and D2 seepage pressures are considered to have played a role in reducing the factor of safety against base sliding.

The validity of applying the back-analysis to event A1 is questionable in view of the high inferred velocity of 40 m/s at the entrance to the run-out zone. As also found by Cruden (1985), the back-calculated run-in friction angle of 3 to 6 degrees is unrealistically low.

Events such as C7, C8 tend towards a Sturzstrom scenario with steep foundation and run-in slopes. Corresponding high values of run-in velocity between 20 and 40 m/s were calculated requiring higher friction angles to meet the observed run-out distance.

The inferred run-in and run-out base friction angles have included within them the average effect of pore water pressures on the effective shear strength of foundation soils for which the effective angles of internal friction is expected to exceed 30 degrees.

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TABLE D-1 - SUMMARY OF WASTE DUMP FAILURE RUN-OUT CHARACTERISTICS

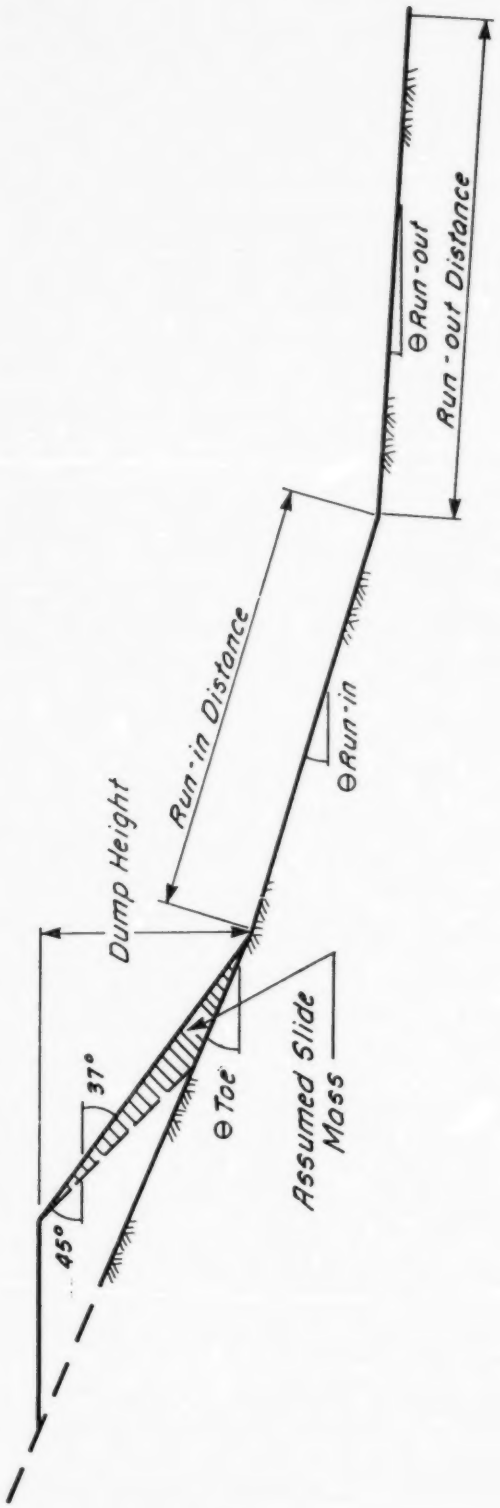
| EVENT NO | LOCATION | DATE | PILE HEIGHT (M) | TOE SLOPE (deg) | VOLUME /M WIDTH (cu.m.) | RUN-IN SLOPE (deg) | RUN-IN DIST (m) | RUN-OUT SLOPE (deg) | RUN-OUT DIST (M) | INFERRED OR OBSERVED VELOCITY (m/s) | OBSERV POINT | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| A1 | | Sept | 225 | 27 | 6600 | 13 | 600 | 7 | 1630 | 40 | | Sturzstrom |
| B1 | | July | 150 | 30 | 2300 | 25 | 270 | 7 | 1965 | | | |
| C1 | | Nov | 200 | 25 | 4700 | 22 | 250 | 3 | 230 | | | |
| C2 | | Nov | 228 | 21 | 6800 | 15 | 225 | 7 | 120 | | | |
| C3 | | July | 230 | 15 | | 14 | | 14 | 200 | 1 to 2 | | Excess pore water pressure |
| C7 | | June | 240 | 27 | 4000 | 22 | 400 | 15 | 350 | | | |
| C8 | | July | 150 | 27 | 2800 | 25 | 550 | 14 | 250 | | | |
| D1 | | May | 170 | 16 | 2600 | 16 | 145 | 8 | 440 | 10-15 | | Excess pore water pressure |
| D2 | | march | 160 | 27 | 2800 | 20 | 455 | 16 | 380 | 15-25 | Run-up | Excess pore water pressure |
| D3 | | June | 150 | 25 | | 19 | 870 | 6 | 360 | 20 | Curve | |
| D4 | | | 150 | 27 | 2500 | 18 | 1370 | 8 | 2120 | | | |
| D5 | | July | 200+ | 26 | | 24 | 200 | 6 | 1040 | | | |

TABLE D-2 - SUMMARY OF RESULTS OF BACK-ANALYSIS OF PAST WASTE DUMP FAILURE EVENTS

| EVENT NO | BEST FIT BACK ANALYSIS RUN-IN VELOCITY (m/s) | RUN-IN FRICTION (deg) | FLOW DEPTH (m) | RUN-OUT FRICTION (deg) | INFERRED OR OBSERVED VELOCITY (M/S) | REMARKS |
|---|---|---|---|---|---|---|
| A1 | 40 - 50 | 6 - 3 | 11 | 13 - 16 | 40+ | Deduced from run-up over |
| B1 | 20 - 30 | 15 - 5 | 10 | 9 - 10 | | |
| C1 | 20 - 30 | 20 -15 | 20 | 16 - 25 | | |
| C2 | 10 - 20 | 15 - 12 | 30 | 18 - 33 | | |
| C3 | 2 | 13 | 20 | 16 | 1 - 2 | observed |
| C7 | 20 - 40 | 22 - 21 | 5 - 15 | 22 - 37 | | |
| C8 | 20 - 4- | 24 - 20 | 5 | 24 - 33 | | |
| D1 | 20 - 30 | 13 - 10 | 10 | 10 - 15 | 10 - 15 | inferred from super-eleva of run-out |
| D2 | 20 - 40 | 20 - 15 | 6 | 15 - 33 | 15 - 25 | inferred from run-up onto opposite valley slope |
| D3 | 20 | 18 | 10 | 14 | 20 | inferred from super-eleva of run-out |
| D4 | 50 - 90 | 15 - 10 | 5 | 15 - 27 | | |

**GEOMETRIC PARAMETERS FOR ANALYSIS OF WASTE DUMP RUN-OUT**

Figure **D-1**

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# APPENDIX E

# THREE-DIMENSIONAL STABILITY ANALYSIS

## STABILITY ANALYSES FOR A WASTE ROCK DUMP LOCATED WITHIN A V-SHAPED TOPOGRAPHIC DEPRESSION

When considering the stability of a rock slope in which the joint systems that intersect the rock mass form the boundaries of potential failure wedges, it is necessary to examine the forces that act normal to the wedge boundaries in order to provide an estimate of the forces available to resist potential downslope displacement of the wedge. Where the joint systems that form the boundaries of the potential failure are steeply inclined, the forces acting normal to the wedge boundaries are larger than the vertical component of the forces representing the weight of the wedgte, and the potential shearing resistance on the wedge boundaries is increased. A method of estimating the forces that act normal to the boundaries of potential failure wedges is presented in Hoek and Bray (1977).

When a waste rock dump is located within a V-shaped depression, for example within a steep-sided gully, wedging action results in an increase in the forces that act in a direction normal to the surfaces that form the sides of the gully, i.e. normal to the dump/foundation surface of contact. Since the available shearing resistance along the dump/foundation surface of contact is proportional to the forces that act normal to this surface, it follows that the wedging action of the waste rock within the V-shaped depression serves to increase stability. This principle can be used to advantage during the development of a dump to the stage at which advancement of the dump crest in a direction parallel to contour over a Type-A foundation can commence.

The following paragraphs describe a method for assessing the stability of a segment of a waste rock dump that is supported on a founda-

tion consisting of two planes comprising the side slopes of a gully, typical of the type of terrain that is common in the Cordilleran Region of British Columbia. The stability analyses assume that segregation of the coarsest rock fragments has occurred, attendant with end dumping of the waste rock at the dump crest, and that a zone of coarse rock is extant over the base region of the dump. This zone of coarse rock is assumed to serve as an underdrain which precludes development of surface of contact. It is also assumed that the stability is controlled by the double wedge mechanism. That is, provided the toe wedge remains stable, it will continue to provide support for active wedge in the upper region of the dump face, with the result that the whole of the dump face will remain stable.

Figure E-1 illustrates a waste rock dump that has been developed within a V-shaped topographic depression. The line A-A represents the line of intersection of the two side planes and forms the axis of the draw. If movement of the toe region of this dump were to occur, the direction of potential movement would be parallel to the azimuth of line A-A.

The toe region of this dump can be considered as a series of vertical sections aligned parallel to the line 'A-A'.

For limiting equilibrium of the two-dimensional section (Figure E-2), the applied shearing force tending to destabilize the toe wedge is $P_n (\tan \phi_m)$.

Considering all of the vertical slices comprising the toe wedge for the three-dimensional case, the sum of the normal forces that act in vertical planes aligned parallel to the vertical plane through A-A is

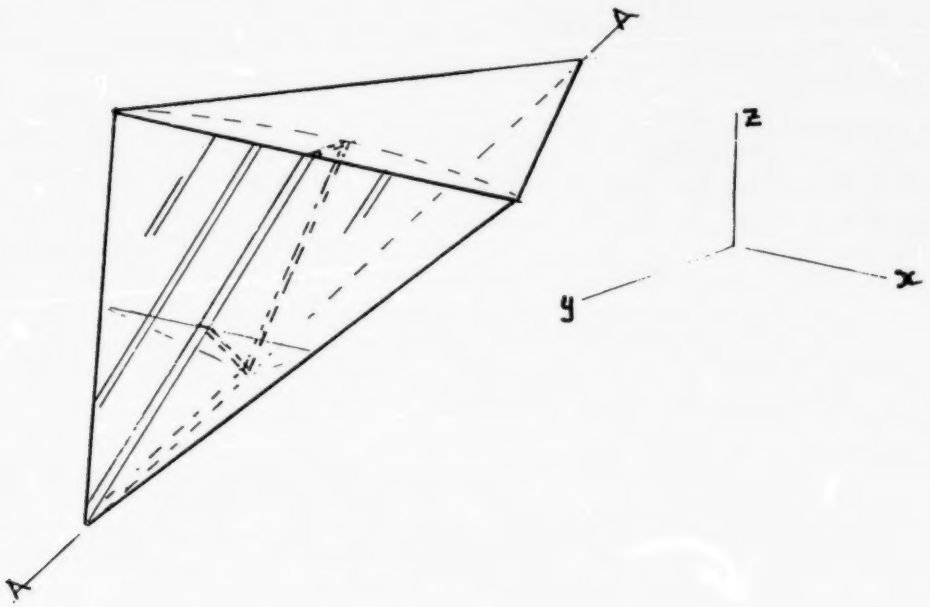
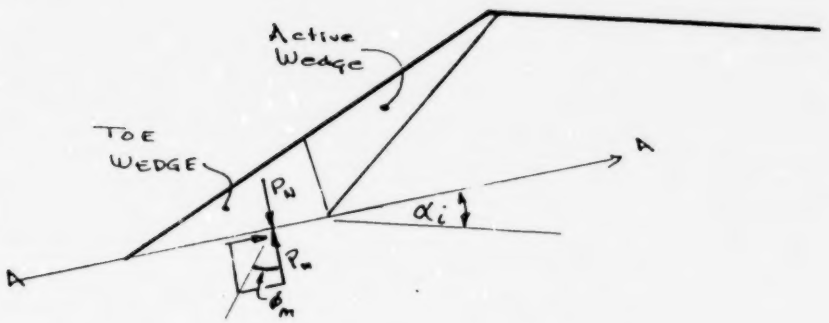



Figure E-1

ISOMETRIC SKETCH

The direction of potential movement
of the toe wedge is restricted to
the azimuth and inclination
of the line of intersection
of the side planes.

FIGURE E-2
VERTICAL SECTION THROUGH A-A




NOTATION

$P_n$ = The force acting normal to the base of the toe wedge

$\phi_m$ = The friction angle mobilized on the base of the toe wedge for limiting equilibrium

$\sum P_n$, and the total of the applied shearing forces that tend to destabilize the toe region of the dump is $\sum P_n \tan\phi_m$.

For limiting equilibrium, the applied shearing force is:

$$P_n (\tan \phi_m).$$

The forces that act in the plane normal to line A-A are shown on Figure E-3. $P_L$ and $P_R$ are the base reaction forces that act in a direction normal to the left and right side planes respectively.

By Application of Simple Statics

$$\Sigma \text{ HOR} = 0, \quad \text{so} \quad P_L \sin\alpha_L = P_R \sin\alpha_R$$

$$\text{or} \quad P_L = P_R \frac{\sin\alpha_R}{\sin\alpha_L} \qquad (1)$$

$$\Sigma \text{ VERT} = 0 \quad \text{so} \ldots \quad P_L \cos\alpha_L + P_R \cos\alpha_R = \Sigma P_N \qquad (2)$$

Substituting (1) and (2)

$$P_R \left( \frac{\sin\alpha_R}{\sin\alpha_L} \cdot \cos\alpha_L + \cos\alpha_R \right) = \Sigma P_N$$

$$P_R = \frac{\Sigma P_N}{\frac{\sin\alpha_R}{\sin\alpha_L} \cdot \cos\alpha_L + \cos\alpha_R} \qquad (3)$$

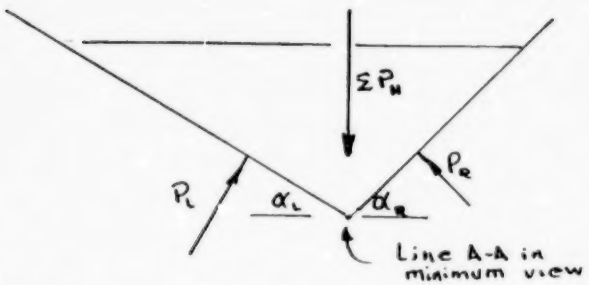



FIGURE E-3
Illustrating the forces that act
in the plane normal to line A-A,
the line of intersection of the side planes.

Substituting (3) into (1) . . . . .

$$P_L = \frac{\Sigma P_N}{\frac{\sin \alpha_R}{\sin \alpha_L} \cdot \cos \alpha_L + \cos \alpha_R} \left( \frac{\sin \alpha_R}{\sin \alpha_L} \right)$$

The available shearing resistance is . . . . $(P_R + P_L) \tan \phi$

where $\phi'$ is the available friction angle for the dump/foundation surface of contact.

$$= \Sigma P_N \left( 1 + \frac{\sin \alpha_R}{\sin \alpha_L} \right) \left( \frac{1}{\frac{\sin \alpha_R}{\sin \alpha_L} \cdot \cos \alpha_L + \cos \alpha_R} \right) \tan \phi'$$

The factor of safety 'F' for the toe wedge = total available shearing resistance divided by total applied shearing forces, or . . . . .

$$F = \frac{\Sigma P_N \left( 1 + \frac{\sin \alpha_R}{\sin \alpha_L} \right) \left( \frac{1}{\frac{\sin \alpha_R}{\sin \alpha_L} \cdot \cos \alpha_L + \cos \alpha_R} \right) \tan \phi'}{\Sigma P_N \tan \phi_m}$$

$$F = \left( 1 + \frac{\sin \alpha_R}{\sin \alpha_L} \right) \left( \frac{1}{\frac{\sin \alpha_R}{\sin \alpha_L} \cdot \cos \alpha_L + \cos \alpha_R} \right) \frac{\tan \phi'}{\tan \phi_m}$$

$$\frac{\tan \phi'}{\tan \phi_m}$$

. . . . . is the expression for the factor of safety with respect to potential downslope movement of the toe wedge for the two-dimensional

case in which the dump/foundation surface of contact can be represented by an inclined plane surface.

Wedging of the waste rock within the topographic V-shaped depression increases the factor of safety with respect to potential down-slope movement of the toe wedge of the dump by the factor . . . .

$$\left(1 + \frac{\sin \alpha_R}{\sin \alpha_L}\right) \left(\frac{1}{\frac{\sin \alpha_R}{\sin \alpha_L} \cdot \cos \alpha_L + \cos \alpha_R}\right)$$

which simplifies to the expression . . . . .

$$\frac{\sin \alpha_L + \sin \alpha_R}{\sin \alpha_L * \cos \alpha_R + \sin \alpha_R * \cos \alpha_L}$$

The three-dimensional factor of safety with respect to potential downslope displacement of the toe wedge as a result of base sliding is the product of the factor of safety for the two dimensional case in which the toe region of the dump is supported on a plane which is inclined at an angle parallel to the inclination of the line of intersection of the two side planes, and the above expression.